

Foundation Investigation and Design Report

*Retaining Wall or Slope Alternative, Station 18+800 to 18+825
Highway 400 Widening from North of King Road to
South of Lloydtown-Aurora Road, King City, Ontario
Assignment No. 2017-E-0016-015, G.W.P. 2835-02-00*

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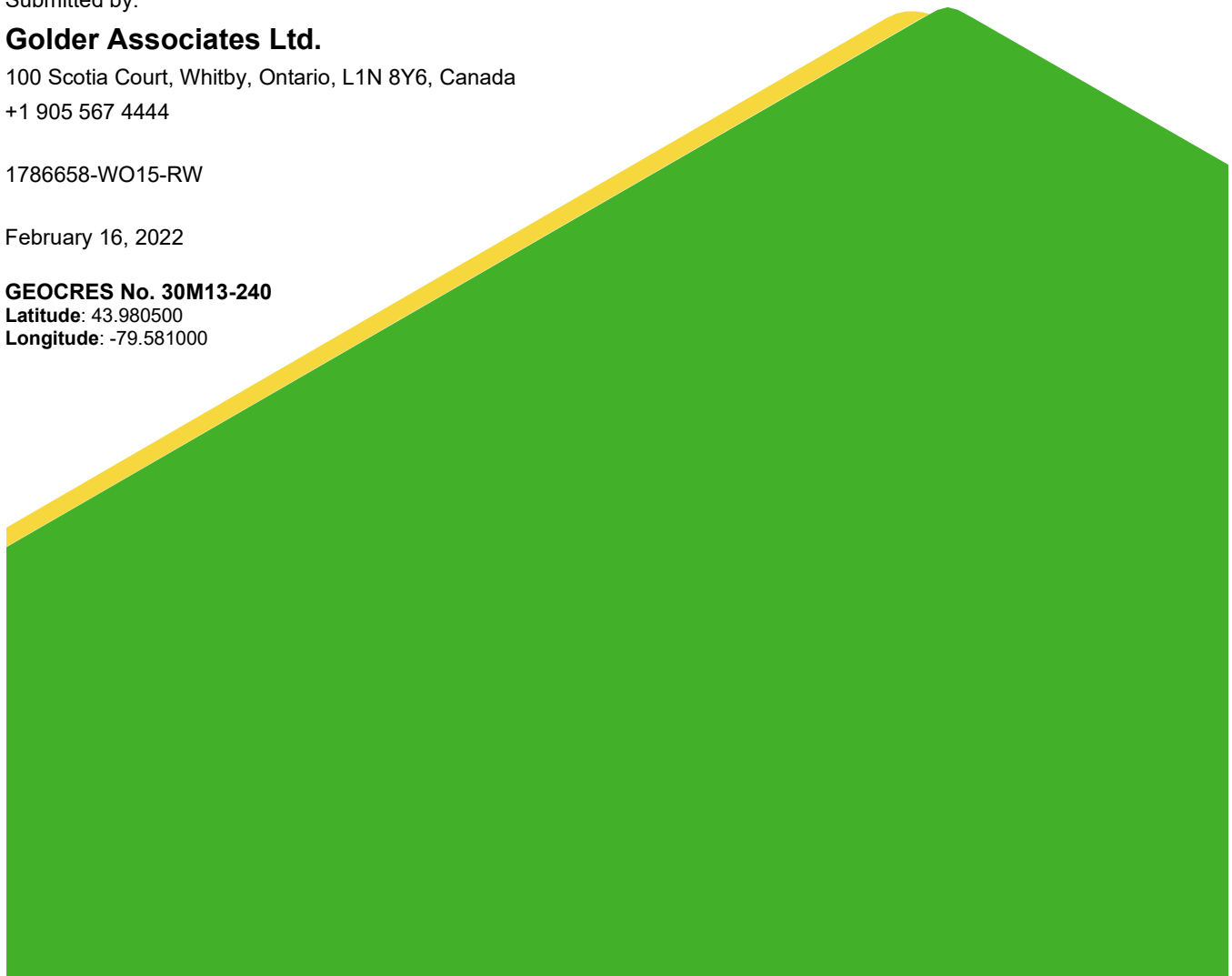
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PART A

FOUNDATION INVESTIGATION REPORT
PROPOSED RETAINING WALL OR SLOPE ALTERNATIVE,
STATION 18+800 TO 18+825
HIGHWAY 400 WIDENING FROM NORTH OF KING ROAD TO
SOUTH OF LLOYDTOWN-AURORA ROAD, KING CITY, ONTARIO
ASSIGNMENT NO. 2017-E-0016-015, G.W.P. 2385-02-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the detail design of the widening of Highway 400 from north of King Road to south of 16th Sideroad, and from north of 16th Sideroad to south of Lloydtown-Aurora Road (i.e., from King Road to Lloydtown-Aurora Road, excluding those portions of the highway widening completed around the 16th Sideroad interchange under a previous contract), as part of MTO Agreement No. 2017-E-0016, Assignment #15.

This report addresses the foundation investigation carried out for a proposed retaining wall or slope alternative located near the toe of the widened Highway 400 embankment on the west side of the southbound lanes of Highway 400 at about Station 18+800, at the approximate location shown on Drawing 1. The purpose of this investigation is to establish the subsurface conditions at the location of the proposed retaining wall or slope based on borehole drilling and geotechnical laboratory testing on selected samples and based on the results of the investigation, provide foundation engineering recommendations for the design and construction of the proposed retaining wall and slope alternatives.

This report was developed based on information from the current foundation investigation, supplemented with a relevant borehole at this site from Golder's previous foundation investigation, presented in the following report:

- **MTO GEOCREC 30M13-214:** "Foundation Investigation and Design Report, Culverts from Station 13+375 to Station 22+500, Highway 400 Widening from North of King Road to South Canal Bank Road, Regional Municipality of York, G.W.P. 2835-02-00", Golder Report Number 09-1111-0018-10, dated December 1, 2015.

2.0 PROJECT / SITE DESCRIPTION

Based on the current design, the proposed retaining wall is located between Station 18+800 and 18+825, near the toe of the proposed widened embankment about 30 m west of the existing west pavement edge of Highway 400 southbound lanes, as shown on Drawing 1. At the proposed retaining wall location, the existing ground surface is at about Elevation 311 m.

The proposed retaining wall is located on the edge of the existing MTO Right-of-Way (ROW). The lands west of the proposed retaining wall consist of marsh lands surrounded by agricultural lands.

An existing culvert (designated as Culvert 36) crosses the highway at about Station 18+810, with the current culvert inlet in the vicinity of the proposed retaining wall location. It is understood that the existing culvert and highway embankment at this location was recently rehabilitated. An access road to the culvert inlet was in place at the time of the current investigation, as shown on Figures 1 and 2 following the text of this report.

It is noted that as part of this MTO assignment, a foundation investigation was carried out for the replacement of Culvert 36; the results of the investigation and the foundation recommendations for the culvert replacement are provided under a separate cover.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigation (GEOCREC 30M13-214)

As outlined in GEOCREC 30M13-214, a foundation investigation was previously carried out for thirteen culvert sites along Highway 400 from north of King Road to south of Canal Bank Road. As part of the previous foundation investigation, one borehole (designated as Borehole C36-1) was advanced at the existing outlet of

Culvert 36, in the vicinity of the proposed retaining wall. The borehole location is shown on the Drawing 1 and the borehole record is provided in Appendix A.

The borehole location and ground surface elevation were surveyed by Callon Dietz Incorporated, Ontario Land Surveyors. The borehole location (in MTM NAD 83 Zone 10 northing and easting coordinates and latitude and longitude), ground surface elevation (referenced to Geodetic datum), and borehole depth are summarized below.

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
C36-1	4,871,168.2 (43.980477)	298,317.5 (-79.580808)	310.5	6.6

3.2 2020 Investigation

The current foundation investigation was carried out in December 2020, during which time two boreholes (designated as Boreholes RW-1 and RW-2) were advanced to depths of 11.3 m and 11.0 m below ground surface. The boreholes were advanced in the vicinity of the proposed retaining wall, as shown on Drawing 1. The borehole records are provided in Appendix A.

The field investigation was carried out using a D-50T track-mounted drill rig, supplied and operated by Walker Drilling Inc. of Utopia, Ontario. The boreholes were advanced using 203 mm outside diameter continuous flight hollow stem augers. Soil samples were generally obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures. The split-spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions.

The groundwater conditions in the open boreholes were observed during the drilling operations and a standpipe piezometer was installed in a separate borehole adjacent to Borehole RW-1 to permit monitoring of the groundwater level at the borehole location. The standpipe piezometer consists of 50 mm diameter PVC pipe, with a slotted screen sealed at a selected depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the screen sand pack were backfilled to the ground surface with bentonite pellets and a stick-up monument casing was provided at the piezometer location. Piezometer installation details and water level readings are described on the borehole record presented in Appendix A. Borehole RW-2, in which a standpipe piezometer was not installed, was backfilled to ground surface with bentonite upon completion, in general accordance with Ontario Regulation 903 (as amended).

The field work was observed on a full-time basis by a member of Golder's engineering staff, who located the boreholes, arranged for the clearance of underground utilities, directed the drilling, sampling and in situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's geotechnical laboratory in Mississauga, Ontario where the samples underwent further visual examination and laboratory testing. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples, in accordance with MTO LS and/or ASTM standards, as appropriate.

One selected soil sample was submitted to a specialist analytical laboratory under chain of custody procedures for testing of conductivity / resistivity, pH and chemical analysis of sulphate and chloride content, to assess the potential for the soil to cause deterioration to buried concrete and corrosion to steel.

The borehole locations and the ground surface elevations were surveyed by Golder using a Trimble Geo7X with a horizontal and vertical accuracy of 0.1 m. The borehole locations and elevations are referenced relative to MTM NAD 83 (Zone 10) northing and easting, and to geodetic datum (HT2_0 / CGVD 1928:1978), respectively. The borehole locations (including northing/easting and latitude/longitude), ground surface elevations, and drilled depths are summarized below.

Borehole No.	MTM NAD83 (Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude,°)	Easting (m) (Longitude,°)		
RW-1	4,871,174.9 (43.980538)	298,297.8 (-79.581054)	311.0	11.3
RW-2	4,871,154.9 (43.980357)	298,295.0 (-79.581088)	310.7	10.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The section of Highway 400 included in this project traverses the three physiographic regions known as the South Slope, Oak Ridges Moraine and Simcoe Lowlands, according to *The Physiography of Southern Ontario* (Chapman and Putman, 1984)¹. The South Slope is present at the southern portion of the project length, extending south from about 2 km north of King Road. The Oak Ridge Moraines is present through the center portion of the project length, extending from about 2 km north of King Road to about 2 km south of Lloydtown-Aurora Road. The Simcoe Lowlands is present at the northern portion of the project length, extending north from about 2 km south of Lloydtown-Aurora Road.

The proposed retaining wall is located within the Oak Ridges Moraine physiographic region. The Moraine represents a ridge of land that runs parallel to and about 60 km north of Lake Ontario, extending from the Niagara Escarpment in the west to the Trent River in the east. The Oak Ridges Moraine predominantly consists of sand and gravel to sand and some silt, although the geology does vary along its length and in the King Township area these soils are often overlain by till. It is understood that during grading for the initial construction of Highway 400 through this area, deep cuts exposed up to about 10 m of till overlying the sand and gravel deposits.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the previous investigation (Borehole C36-1) and the current investigation (Boreholes RW-1 and RW-2) are presented on the borehole records in Appendix A. The results of the geotechnical laboratory tests and analytical laboratory testing carried out as part of the current investigation are presented in Appendices B and C, respectively.

¹ Chapman, L.J. and Putnam, D.F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.

The results of the in situ field tests (i.e., SPT “N”-values) as presented on the borehole records and in Section 4.2 are uncorrected. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations.

4.2.1 Topsoil

A 50 mm thick deposit of topsoil was encountered at ground surface in Borehole C36-1, which was drilled at the west toe of the existing highway embankment.

4.2.2 Silty Sand Fill

A 0.2 m thick layer of silty sand containing trace gravel was encountered underlying the topsoil in Borehole C36-1, extending to a depth of about 0.3 m below ground surface (Elevation 310.2 m). This was interpreted to be fill associated with the toe of the highway embankment based on its location.

4.2.3 CLAYEY SILT (CL)

A 0.7 m and 0.8 m thick surficial layer of clayey silt containing trace sand, trace gravel, trace rootlets and organics was encountered at ground surface in Borehole RW-2 and underlying the silty sand fill in Borehole C36-1. The SPT “N”-values measured within the clayey silt deposit ranged from 0 blows (i.e., weight of hammer, WH) to 16 blows per 0.3 m of penetration, indicating a very soft to very stiff consistency. This layer extends to Elevation 310.0 m to 309.4 m in the two boreholes in which it was encountered.

As part of the previous investigation, Atterberg limits testing was carried out on one sample of the clayey silt and the results are presented on the borehole record in Appendix A. The test measured a liquid limit of about 16%, a plastic limit of about 26%, and a plasticity index of about 10%, indicating the soil is a clayey silt of low plasticity. The natural water content measured on two samples of the clayey silt deposit are about 19% and 42%; this is within the plastic range for the sample from Borehole C36-1, while the higher measured value in the sample from Borehole RW-2 is attributed in part to the presence of organics.

4.2.4 SILT (ML) to SILTY SAND (SM)

A 1.1 m to 3.8 m thick deposit ranging in composition from silt to sandy silt to silty sand, trace gravel, trace clay was encountered at ground surface in Borehole RW-1 and underlying the clayey silt deposit in Boreholes C36-1 and RW-2. The top of this layer was encountered between Elevation 311.0 m and 309.4 m, and it extends to depths ranging from 2.2 m to 4.5 m below ground surface (Elevation 308.3 m to 306.2 m). The deposit contains trace organics and rootlets to depths of 1.4 m and 1.5 m in Boreholes RW-1 and RW-2, respectively. The SPT “N”-values measured within the silt to silty sand deposit range from 4 blows to 25 blows per 0.3 m of penetration, indicating the deposit is loose to compact.

As part of the current investigation, grain size distribution testing was carried out on three samples of the silt to silty sand deposit and the results are presented on Figure B-1 in Appendix B. Atterberg limits testing was carried out on one sample and the results are presented on Figure B-2 in Appendix B. The Atterberg limits testing measured a liquid limit of about 15%, a plastic limit of about 13%, and a plasticity index of about 2%, indicating the fines portion of the sample consists of silt of slight plasticity. In general, the silt to silty sand deposit ranges from non-plastic to slightly plastic. The natural water contents measured on samples of the silt to sand deposit range from about 15% to 22%. The organic content of two upper samples of the silt to sand deposit is about 1%.

4.2.5 SILT and Sand to Sandy SILT (ML) to CLAYEY SILT (CL) Till

A non-cohesive to cohesive till deposit was encountered underlying the silt to silty sand deposit in all boreholes, at depths ranging from 2.2 m and 4.5 m below ground surface (Elevations 308.3 m to 306.2 m). The till deposits extended to the borehole termination depths which are between 6.6 m and 11.3 m (Elevations 303.9 m and 299.7 m). The till deposit varies from a non-cohesive to slightly plastic silt and sand to sandy silt, trace gravel, trace clay to a cohesive clayey silt till, some sand, trace clay.

The SPT “N”-values measured within the non-cohesive to slightly plastic portion of the till deposit range from 10 blows to 105 blows per 0.3 m of penetration, with one SPT “N”-value of 100 blows per 0.15 m, indicating the deposit is compact to very dense. One SPT “N”-value of 90 blows per 0.3 m of penetration was measured within the cohesive portion of the till deposit, suggesting a hard consistency.

Grain size distribution testing was carried out on three samples of the till deposit from the current investigation and the results are presented on Figure B-3 in Appendix B. Atterberg limit testing was carried out on two samples of the till deposit and the results are presented on Figure B-4 in Appendix B. The Atterberg limits test on the silt and sand till sample measured a liquid limit of about 12%, a plastic limit of about 10% and a plasticity index of about 2% while the test on the cohesive portion of the till measured a liquid limit of about 19%, a plastic limit of about 11%, and a plasticity index of about 8%. Together with field observations, this indicates the fine fraction of the silt and sand to sandy silt till is non-plastic to slightly plastic and confirms that the deposit grades to clayey silt of low plasticity. The natural water content measured on six samples of the till deposit range from about 8% to 9%.

4.2.6 Groundwater

Details of the groundwater levels measured in the open boreholes during and/or on completion of drilling and in the piezometer installed in Borehole RW-1 are presented on the borehole records in Appendix A and as summarized below. These observations are based on the conditions at the time of drilling and are not necessarily representative of the stabilized groundwater level at the site.

Borehole No.	Depth to Groundwater (m)	Groundwater Elevation (m)	Date of Observation	Notes
C36-1	2.1	308.4	November 25, 2010	Depth encountered during drilling
RW-1	1.5	309.5	December 4, 2020	Depth observed during sampling and measured on completion of drilling
	0.6	310.4	February 10, 2021	Depth measured in piezometer screened in silt to silty sand deposit
RW-2	0.9	309.8	December 4, 2020	Depth observed during sampling and measure on completion of drilling

Based on the above-noted observations and the water level measured in the piezometer, it is anticipated that the groundwater level associated with the silt to silty sand deposit will be up to approximately Elevation 310.5 m in “drier” periods of the year, just below the existing ground surface in the proposed retaining wall area. The groundwater level at the site will be subject to seasonal fluctuations and should be expected to be higher during the spring season or during and following periods of heavy precipitation. Based on the vegetation present at the

site and mapping of this area as a marsh, it should be expected that the groundwater level may be at the ground surface (i.e., at approximately Elevation 310.8 to 311 m) during wetter periods of the year.

4.3 Analytical Testing Results

One soil sample was submitted for analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. Detailed analytical test results are included in Appendix C and the results are summarized below.

Borehole No. / Sample No.	pH	Resistivity (ohm-cm)	Electrical Conductivity (umho/cm)	Chlorides (ug/g)	Soluble Sulphates (ug/g)
RW-1 / 3	7.9	13,000	78	<20 (Below RDL)	<20 (Below RDL)

Note:

1. RDL indicates "Reportable Detection Limit" of 20 µg/g

5.0 CLOSURE

This Foundation Investigation Report was prepared by Anastasia Poliacik, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., an MTO Foundations Designated Contact and Principal of Golder, conducted an independent technical and quality control review of the report.

Signature Page

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PART B

**FOUNDATION DESIGN REPORT
PROPOSED RETAINING WALL OR SLOPE ALTERNATIVE,
STATION 18+800 TO 18+825
HIGHWAY 400 WIDENING FROM NORTH OF KING ROAD TO
SOUTH OF LLOYDTOWN-AURORA ROAD, KING CITY, ONTARIO
ASSIGNMENT NO. 2017-E-0016-015, G.W.P. 2385-02-00**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides detail foundation recommendations for the design of a proposed retaining wall or slope alternative to be constructed as part of the Highway 400 widening from north of King Road to south of 16th Sideroad, and from north of 16th Sideroad to south of Lloydtown-Aurora Road. The proposed retaining wall or slope alternative is located along the western limit of the MTO right-of-way extending from approximately Station 18+800 to 18+825, at the west end of Culvert C36. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced as part of a foundation investigation at and near the proposed retaining wall location.

This Foundation Design Report with its interpretation and recommendations are for the use of MTO and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A of the report (Foundation Investigation Report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Contractors must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Retaining Walls and Slopes - Foundations Options

From a foundation perspective, the following retaining wall and slope types are considered feasible alternatives. A summary comparison of feasible retaining wall and slope options based on geotechnical/foundations-related advantages, disadvantages, relative costs, and risks/consequences is presented in Table 1, following the text of this report.

- **Shallow Foundations – Concrete Cantilever Wall:** Shallow foundations comprised of strip footings supported on the compact native soils or on a compacted Granular 'A' pad are considered feasible for support of the retaining wall at this site. This wall type is considered the preferred option given the relatively limited depth of excavation required to achieve the required geotechnical resistances. This option would require dewatering of the surficial silt to silty sand deposit for construction of the footing in "dry" conditions.
- **Shallow Foundations – Reinforced Soil System (RSS) Wall:** An RSS wall with the front facing supported on shallow strip footing is geotechnically feasible for the proposed retaining wall; however, the use of reinforced soil systems does present risks if used adjacent to creeks/drainage channels where there is potential for flooding which could contribute to loss of granular backfill in the reinforced zone and this option would require project-specific approval by MTO's RSS Committee. If this option were permitted and approved by the MTO RSS Committee, it is anticipated that the retaining wall would be constructed in conjunction with the highway embankment widening, so temporary protection systems would not be required. This option would still require dewatering; however, excavations could be shallower than for concrete retaining wall footings which must extend below the frost depth.
- **Deep Foundation – Driven Piles:** A concrete retaining wall could be supported on driven H-piles or tube piles that extend into the dense to very dense till deposit. However, the pile cap for such a configuration would need to be founded below frost depth, and hence excavations would be similar to those needed for a concrete retaining wall on shallow foundations, affording no advantage related to dewatering requirements.

As the soils provide adequate geotechnical resistances for design of shallow foundations, driven piles are not a preferred solution at this site.

- **Deep Foundations – Secant Pile Wall:** Drilled shaft (caisson) walls founded in the very dense / hard glacial till deposit may be considered for the proposed retaining wall, although this solution is not considered necessary given that reasonable founding conditions are available for shallow strip footings as described above. This wall type consists of primary (king) and secondary caissons at an appropriate diameter and embedment length to provide the required axial and lateral resistances for the retained soil behind the wall and other surcharge loads applicable to the wall design. Soil anchors can be used to provide additional lateral stiffness to maintain horizontal movement within tolerable limits, if necessary. Liners would be required during the drilled shaft installation to control the wet, non-cohesive soils and groundwater. This option would likely require more significant working pad preparation to support the caisson rig, as compared with the conventional construction equipment that would be used for shallow foundation construction.
- **Deep Foundations – Soldier Piles with Reinforced Concrete Facing Panels:** A soldier pile and panel system (including proprietary cast-in-place and pre-cast panel wall systems) may be considered for the proposed retaining wall, although such wall types tend to be used in top-down (cut) construction rather than at sites such as this where the wall would retain new fill placed for the embankment widening. Soil anchors would likely be required to provide adequate lateral stiffness for the wall heights and sloping ground conditions at this site. This option would require some degree of groundwater management, although likely less than for a concrete retaining wall footing. This option would permit installation of composite drainage panels or other measures for resisting or controlling the groundwater and frost pressures behind the concrete panels. Similar to the other deep foundation option above, this option would likely require a more significant working pad to support the caisson pile rig.
- **Retained Soil System (RSS) Slope:** An RSS slope inclined at 1.5 Horizontal to 1 Vertical (1.5H:1V) with a reinforced zone extending 10 m behind the slope face is geotechnically feasible at this site and has an appropriate factor of safety for global stability; however use of reinforced soil systems does present risks if used adjacent to creeks/drainage channels where there is potential for flooding which could contribute to loss of granular backfill in the reinforced zone, and similar to the RSS wall, this option would require project-specific approval by MTO's RSS Committee. If approved, this option would require subexcavation of the soft to very stiff silty clay to clayey silt deposit, extending to Elevation 309.4 m (about 1 m below ground surface). Although this option would require dewatering, excavations and dewatering requirements would be less than for the retaining wall options outlined above.
- **Rock Fill Slope:** A rock fill slope inclined at 1.5H:1V is geotechnically feasible at this site and has an appropriate factor of safety for global stability. Use of rock fill adjacent to creeks/drainage channels would require use of geotextile between the rock fill and native soil and existing or new embankment fill, to prevent loss of fine soil particles into voids in the rock fill associated with changes in the water level/flooding in the vicinity of the culvert opening. This option would require subexcavation of the soft to very stiff silty clay to clayey silt deposit, extending to Elevation 309.4 m (about 1 m below ground surface). Although this option would require dewatering, excavations and dewatering requirements would be less than for the retaining wall options outlined above. Given the local geology, rock fill would have to be imported to site for use. Considering the relatively short length of the embankment area which requires reinforcement (about 25 m in length) and the costs for importing rock fill material, this option may not be a practical alternative.

Based on a comparison of the advantages/disadvantages between the various wall and slope types and supporting foundation alternatives presented in Table 1 and described above, and given the subsurface conditions encountered in the boreholes, from a geotechnical/foundations perspective, the preferred alternatives are a concrete cantilever wall, an RSS slope, or a rock fill slope. RSS walls, while geotechnically feasible for the subsurface conditions at this site, are generally not preferred by MTO adjacent to creek/drainage channels with potential flooding conditions. Deep foundations are not considered necessary at this site given that the shallow subsoils provide adequate geotechnical resistance for the retaining wall, and therefore deep foundation options are not discussed further in this report.

It should be noted that the selection of the type of walls / slopes and foundation alternatives will also depend on factors beyond foundation recommendations.

6.3 Design Considerations

6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2019 *Canadian Highway Bridge Design Code* CAN/CSA S6:19 (CHBDC, 2019) and its *Commentary*, the retaining wall and its foundation system may be classified as geotechnical systems designed for application along a transportation corridor with large traffic volumes and with potential impacts on other transportation corridors, resulting in a “typical consequence level” associated with exceeding limit states design. In addition, given the project-specific foundation investigation carried out at this site (as presented in Part A of the report), in comparison to the degree of site understanding in Section 6.5 of CHBDC (2019), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC (2019) have been used for design.

6.3.2 Seismic Design – Seismic Site Class

A conservative site classification is based on physical borehole information obtained at depths of less than 30 m and based on general knowledge of the local geology and physiography. The SPT “N”-values measured in the soil deposits and the interpreted shear wave velocity of soils up to 30 m below founding level are used to define the seismic site classification. Although the foundation investigation does not extend 30 m below the founding level, based on this methodology, it is considered that a Site Class D ($15 < N_{60} < 50$) would be applicable for the design at this site in accordance with Table 4.1 of the CHBDC (2019), and in the absence of any geophysical testing.

To determine the actual site classification based on physical on-site measurements of shear wave velocity as required by the 2012 OBC, the Multichannel Analysis of Surface Waves (MASW) can be utilized. However, given that this project is located in an area of low seismicity and low seismic hazard, it is not considered necessary to complete MASW testing at this site to refine the design. It is noted that a higher (improved) Site Class is not necessarily guaranteed.

6.3.3 Seismic Design – Liquefaction Potential

Based on the degree of compaction and fines content of the generally compact silt to silty sand deposit and the compact to very dense/hard till below the retaining wall, and the site-specific peak ground acceleration, the soils at this site are considered to have a low potential for liquefaction during a design seismic event. This will exceed

the requirements of CHBDC Table 4.1 and Clause 6.14.2.3 (i.e., major-route geotechnical systems to have at least 50% of travelled lanes available for use following ground options with a return period of at least 475 years).

6.4 Retaining Walls

6.4.1 Proposed Retaining Wall Details

Based on the current design, the proposed retaining wall details are summarized in the table below. It is understood that the proposed retaining wall will be constructed as part of the highway embankment widening and will not require excavations through the highway embankment (i.e., will not require temporary protection systems during staging). The wall will be constructed around the west end of the new 1,700 mm diameter pipe culvert (Culvert 36) that is being placed as part of the westward widening; this pipe has its invert at Elevation 309.5 m. The top of the wall is proposed to decline from about Elevation 312.2 m at the north end of the wall (Station 18+825) to about Elevation 311.5 m at the south end of the wall. It is understood that the ground surface behind/above the wall will be oriented at 2H:1V.

Length (m)	Maximum Wall Height (m)	Founding Elevation (m)	Final Ground Surface Elevation in Front of Wall (m)	Final Ground Elevation Behind Wall (m)
25	3.9	308.3	309.8 to 310.2	Sloped upward at 2H:1V to 317.5

6.4.2 Concrete Cantilever Wall

6.4.2.1 Founding Elevations

The proposed retaining wall can be founded on conventional strip (shallow) foundations bearing on the compact silt to silty sand deposit following removal of any topsoil/organic soils, soft clayey silt or other loose / soft / deleterious soils within the foundation footprint. All footings should be founded at a minimum depth of 1.5 m below the adjacent final grade to provide adequate protection against frost penetration, in accordance with OPSS 3090.101 (*Foundation, Frost Penetration Depths for Southern Ontario*).

In order to extend below topsoil/organics, soft clayey silt and loose soils, the retaining wall footing should be founded at or below Elevation 309.5 m. However, it is understood that the proposed finished grade in front of the wall will be at approximately Elevation 309.8 m, rising slightly toward the north end of the wall; therefore, the retaining wall footing should be founded at Elevation 308.3 m for adequate frost protection. This founding level could be stepped higher where the finished grade in front of the wall rises, provided that the stepped footing remains founded below Elevation 309.5 m.

The footing subgrade should be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that any unsuitable material has been removed. Where sub-excavation of unsuitable materials is required, the sub-excavated area should be backfilled with granular material meeting OPSS.PROV 1010 Granular 'A' or Granular 'B' Type II that is placed and compacted in accordance with OPSS.PROV 501 (*Compacting*).

The fine-grained subgrade will be susceptible to disturbance and degradation on exposure to water and construction traffic. Therefore, it is recommended that a minimum 100 mm thick concrete working slab be placed over the subgrade to protect the integrity of the foundation soils. A Special Provision should be included in the Contract Documents to address the working slab (see Appendix E).

6.4.2.2 Geotechnical Resistance

Strip footings constructed on the properly prepared subgrade, at or below the design elevation given in Section 6.4.2.1, should be designed based on the factored ultimate geotechnical resistance and the factored serviceability geotechnical resistance (for 25 mm of settlement) given below.

Proposed Founding Elevation (m)	Footing Size (Width x Length) (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
308.3 (or a minimum of 1.5 m below adjacent ground surface provided footing is maintained below Elevation 309.5 m)	5 x 25	550	250

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width and founding elevation and as such, the geotechnical resistances should be reviewed if the footing width or founding elevations vary from that specified above.

The factored ultimate geotechnical resistances provided are based on a load applied concentrically to the centreline/centroid of the footing, as shown on Figure 6.4 of the CHBDC (2019). Where a load is applied eccentrically from the centreline/centroid of the footing, the pressure distribution and the eccentricity limit of the footing should be taken into consideration in accordance with Section 6.10.5 of CHBDC (2019) and its *Commentary*. Once the structural design is substantially complete, the structural engineer should verify with Golder whether the factored ultimate and serviceability geotechnical resistances provided above require review based on load inclination.

6.4.2.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the concrete footing and the subgrade should be calculated in accordance with Section 6.10.4 of the CHBDC (2019). The coefficient of friction, $\tan \phi'$, for a cast-in-place concrete footing or working slab on the native cohesive soils are presented below. The unfactored coefficient of friction between a cast-in-place concrete footing and a cured concrete working slab is as interpreted from Naval Facility Engineering Command (NAVFAC, 1986).

Subgrade Material	Unfactored Coefficient of Friction, $\tan \phi'$ or $\tan \delta$
Cast-in-place concrete footing or working slab on compact silt to silty sand	0.6
Cast-in-place concrete footing on concrete working slab	0.7

6.4.3 Retained Soil System (RSS) Walls

Mechanically-reinforced soil retaining systems (retained soil system or RSS walls) could be used at this site based on the subsurface conditions; however, the use of RSS walls adjacent to creeks/drainage channels would require review and endorsement by MTO's RSS Committee. Geotechnical recommendations are provided in the following sections as a basis for design of such an option, but it is noted that this wall type has not been put

forward to the Committee or endorsed at this time as a concrete retaining wall has been selected as the preferred option for this site. The following sub-sections are provided for information purposes.

6.4.3.1 Founding Elevations

A typical RSS wall has a front facing panel system that is supported on a strip footing placed at a shallow depth below the ground surface in front of the wall. The facing footing should be placed within a 500 mm thick levelling pad comprised of OPSS.PROV 1010 Granular 'A', placed in accordance with OPSS.PROV 501 (Compacting), as detailed in Figure 5.2 of MTO's *RSS Wall Design Guidelines* (September 2008). There should be a minimum of 300 mm of Granular 'A' below the facing footing. The compacted granular levelling pad should extend at least 1 m beyond the outside edge of the facing footing, then downward and outward at 1H:1V.

As shown on Figure 5.22 of MTO's *RSS Wall Design Guidelines*, it is recommended that the underside of the levelling pad be founded at a minimum depth of 1 m below the finished grade at the base of the RSS wall. The Granular 'A' levelling pad/facing footing and reinforced soil mass are recommended to be founded no higher than Elevation 309.5 m based on the borehole results. However, based on the proposed finished grade in front of the wall at approximately Elevation 309.8 m, it is anticipated that the levelling pad would be founded at or below Elevation 308.8 m to achieve the minimum embedment depth of 1 m between the underside of the levelling pad and the finished grade.

The subgrade for the RSS wall facing footing/granular pad and reinforced soil mass should be inspected by qualified geotechnical personnel following topsoil stripping and excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that any unsuitable material has been removed. Where sub-excavation of fill or unsuitable materials is required, the sub-excavated area could be backfilled with granular material meeting OPSS.PROV 1010 Granular 'A' or Granular 'B' Type II that is placed and compacted in accordance with OPSS.PROV 501 (*Compacting*).

6.4.3.2 Geotechnical Resistance

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass, the factored ultimate and serviceability geotechnical resistances given below may be used for assessment of the reinforced mass founded on the proof-rolled subgrade at the highest founding elevations provided in Section 6.4.3.1. The recommended minimum strip length is based on that required to satisfy global stability given the sloping ground above the top of the wall (as discussed further in Section 6.5).

Maximum Wall Height / Reinforced Soil Mass Height (m)	Recommended Minimum Strip Length (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
3.4 / 2.9	6.0	500	200

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the subgrade/facing footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be considered in accordance with Sections 6.10.2 of the *CHBDC* (2019).

6.4.3.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the granular levelling pad of the RSS wall and the subgrade should be calculated in accordance with Section 6.10.4 of the *CHBDC* (2019). The coefficient of friction, $\tan \phi'$, for the granular levelling pad of the RSS wall on the properly prepared subgrade may be taken as summarized below.

The coefficient of friction value should be reviewed and revised, if necessary, by the proprietary RSS wall designer.

Subgrade Material	Friction Factor, $\tan \phi'$
Compacted Granular 'A' on native compact silt to silty sand of slight plasticity	0.6

6.4.4 Lateral Earth Pressures for Design

The lateral earth pressures acting on the retaining wall will depend on the type and method of placement of backfill materials, the nature of the soils behind the backfill, the magnitude of the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the wall.

The following recommendations are made concerning the design of the wall:

- Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II should be used as backfill behind the wall. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements should be in accordance with OPSS 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and OPSS 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall, in accordance with CHBDC (2019) Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.
- For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:<1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.31(b) of the *Commentary to the CHBDC* (2019).

If the wall support allows for lateral yielding, active earth pressures may be used in the geotechnical design of the retaining wall under static loading conditions. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.12 of the *Commentary of the CHBDC* (2019). If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

For the retaining wall at this site, it is assumed that the unrestrained case is applicable. The following lateral earth pressure parameters (unfactored) may be used. Values have been provided for level ground above the top of the wall, as well as for a 2H:1V slope above the top of the wall. If the inclination of the slope above the wall differs, appropriate lateral earth pressure coefficients will need to be calculated.

Parameter	Unrestrained Walls – Granular Backfill Per CHBDC Figure C6.31(b)	
	Granular A	Granular B Type II
Soil unit weight (kN/m ³)	22	21
Effective friction angle (°)	35	35
Coefficients of static lateral earth pressure for level ground behind wall: Active, K_a At rest, K_o	0.27 0.43	0.27 0.43
Coefficients of static lateral earth pressure for 2H:1V slope above wall: Active, K_a At rest, K_o	0.39 0.62	0.39 0.62

6.5 Global Stability

Two-dimensional limit equilibrium slope stability analyses were performed using the commercially available program Slide2 (Version 9.017), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis and analysing for both circular and non-circular slip surfaces. For all analyses, the Factors of Safety of numerous potential failure surfaces were computed to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analyses, the Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ , and the geotechnical resistance factor, ϕ_{gu} . (i.e., $FoS = 1/(\Psi \cdot \phi_{gu})$) as defined in the Canadian Highway Bridge Design Code (CHBDC, 2019).

The following minimum Factors of Safety have been targeted for the design of the retaining wall and slope alternatives, as per Table 6.2 of the CHBDC, 2019:

- 1.3 for temporary (undrained) conditions; and
- 1.5 for long-term (drained) conditions.

The following parameters have been used in the stability analyses, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Stratigraphic Unit	γ (kN/m ³)	ϕ' ($^{\circ}$)	S_u (kPa)
New embankment fill (earth fill)	20	33	-
New embankment fill (rock fill, if such an option is adopted)	19	40	-
Existing embankment fill	20	32	-
Very soft to very stiff clayey silt	19	28	25
Loose to compact silt to silty sand	20	30	-
Compact to very dense sandy silt to silt and sand till / hard clayey silt till	21	35	-

For the retaining wall options, a maximum retained wall height of 3.8 m was assumed and a slope inclination of 2H:1V was assumed above the walls. For the slope options, a slope inclination of 1.5H:1V was assumed. A design groundwater level at approximately Elevation 311 m was used in the analyses, slightly above that measured in the piezometer installed in Borehole RW-1 to account for seasonal variation.

The results of the stability analyses are summarized in the table below and indicate that the proposed retaining wall and slope options will have a Factor of Safety equal to or greater than the target values against deep-seated global failure. As shown on Figures D-3 to D-6, the minimum strip length required to achieve the target Factor of Safety (FoS) values for the RSS wall and RSS slope are 6 m and 10 m, respectively.

Retaining Wall / Slope Type	FoS (Temporary)	FoS (Long-Term)	Figures
Concrete Cantilever Wall	1.5	1.6	Figures D1 and D2
RSS Wall	1.4	1.5	Figures D3 and D4
RSS Slope	1.4	1.5	Figures D5 and D6
Rock Fill Slope	1.6	1.6	Figures D7 to D9

If reinforced or rock fill slopes steeper than 1.5H:1V are required to fit the available space for the westward widening of the Highway 400 embankment, further assessment of global stability will be required to ensure the Factor of Safety for the long-term condition is maintained at or above 1.5 or to be granted an exception. For a steeper RSS slope, longer reinforcing strips would likely be necessary to achieve a Factor of Safety of 1.5 or greater in long-term conditions.

6.6 Construction Considerations

6.6.1 Temporary Excavations

It is anticipated that temporary excavations will extend through very soft to very stiff clayey silt, loose to compact silt to silty sand, and potentially into the silt and sand to silty sand till. Excavations must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities (Ontario Regulation 213). The very soft to soft clayey silt and the non-cohesive site soils may be classified as Type 3 and 4 soils, respectively. According to the OHSA, temporary excavations within Type 3 soils

should be made with side slopes 1H:1V or flatter and temporary excavations within Type 4 soils should be made with side slopes 3H:1V or flatter unless dewatering is in place. Depending on the time of year during construction, localized flattening of temporary excavations may be required in some areas during construction.

Excavated material must be stockpiled at a distance away from the excavation equal to or greater than the depth of the open cut excavation. Where sufficient space is not available to stockpile the excavated material at the site, off-site disposal of the excavated material intended for reuse would need to be arranged. Care must also be taken during excavation to ensure that adequate support is provided for any existing structures, roadways and underground services located adjacent to the excavations.

The temporary excavations should be carried out in general accordance with OPSS 902 (*Excavating and Backfilling - Structures*).

6.6.2 Temporary Protection Systems

It is anticipated that temporary protection systems may not be required at this site, as the retaining wall can be constructed in advance of the high fill embankment widening in this area and the slope alternatives can be constructed as part of the high fill embankment widening. However, a sheet pile cut-off wall may assist in groundwater control at this site, as well as support excavation side slopes for retaining wall construction. This section provides brief discussion regarding temporary protection systems for information for MTO and their Contract Administrator.

If required, temporary protection system should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*), as amended by SP 105S09. As the retaining wall is located away from the highway embankment, structures and utilities, the lateral movement should meet Performance Level 3.

For conceptual purposes, sheet pile systems or soldier pile and lagging systems are considered feasible at this site; a sheet pile system could also cut off groundwater infiltration through the excavation sides, reducing the dewatering requirements. Sheet piles should be able to penetrate into the upper portion of the till but it would be difficult to advance them into the denser portions of the till deposit. The presence of cobbles / boulders / gravelly soils within the till deposit could also impede installation of the temporary protection systems, although pre-drilling and/or removal of localized obstructions to facilitate construction of the temporary protection systems is considered feasible. A raker system could be considered for lateral support where required.

The selection and design of any temporary protection system for this site will be the responsibility of the Contractor; however, the following parameters are provided for consideration. The system must be designed to accommodate the loads applied from earth pressures, water pressures and surcharge pressures from area, line or point loads, as well as the effects of sloping ground behind the system. The loading from construction equipment as well as any material stockpiles within a distance defined by a 1 horizontal to 1 vertical line drawn from the bottom of the excavation to the existing ground surface should be included as a surcharge in the design of the temporary protection system.

Stratigraphic Unit	Unit Weight of Material, γ (kN/m ³)	Angle of Internal Friction, ϕ (°)	Undrained Shear Strength, S_u (kPa)	Coefficients of Static Lateral Earth Pressure		
				Active, K_o	At Rest, K_a	Passive, K_p
New embankment fill (earth fill)	20	32	-	0.47	0.31	3.25
Existing embankment fill	20	32	-	0.47	0.31	3.25
Loose to compact silt to silty sand	20	30	-	0.50	0.33	3.00
Compact to very dense sandy silt to silt and sand till / hard clayey silt till	21	35	-	0.43	0.27	3.69

Notes:

1. The lateral earth pressure coefficients presented above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are expected, the coefficients shown need to be corrected accordingly.
2. The total passive resistance below the base of the excavation (i.e., within the shored excavation and / or adjacent to the temporary protection system) may be calculated based on the value of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.27 of the CHBDC (2019) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

6.6.3 Groundwater and Surface Water Control

Considering excavations for concrete cantilever walls, RSS walls, and RSS / rock fill slopes will extend to about Elevation 308.3 m, 308.5 m, and 309.4 m, respectively, and the groundwater level at the site was measured to be up to about Elevation 310.5 m, excavations for the proposed retaining wall will extend about 2 m below the groundwater level and excavations for the slope alternatives will extend about 1 m below the groundwater level. Therefore, dewatering of the silt to silty sand deposit will be required to allow for excavation and construction in dry conditions, in addition to temporary flow passage for the culvert channel. Dewatering operations should be carried out in accordance with OPSS 902 (*Excavation and Backfilling – Structures*) and OPSS.PROV 517 (*Dewatering*) as amended by FOUN0003 (*Dewatering Structure Excavation*), a copy of which is included in Appendix E. It is recommended that the monitoring well in Borehole RW-1 be left in place for the contractor to check the groundwater level at the time of construction.

The groundwater table in the silt to silty sand deposit should be lowered to approximately 1 m below the subgrade level; the underlying till deposit may limit the practical level to which the groundwater can be lowered below the foundation subgrade in some areas. Based on the fine-grained nature of the silt to silty sand deposit and the required drawdown on the order of 2 m to 3 m, it is anticipated that a system of eductors would be necessary, potentially in conjunction with local sumps in areas where drawdown is limited by the underlying till interface. The use of widely-spaced sump pits and pumps, shallow drilled wells with submersible pumps, or drainage that relies on gravity flow of water may not be adequate to lower the groundwater level below subgrade level. The dewatering volumes could be reduced if driven interlocking sheet piles are used to cut off the sides of the excavation.

Water takings in excess of 50,000 L/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater and stormwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MECP Environmental Activity and

Sector Registry (EASR). Registry on the EASR replaces the need to obtain a PTTW for water taking and a Section 53 approval for discharge of water to the environment. A “Water Taking Plan” and a “Discharge Plan” are required by the MECP if water is taken in accordance with an EASR. In all cases, discharge under the EASR must be in accordance with a Discharge Plan (to be developed by a qualified professional). The Contractor will be responsible for obtaining any required discharge approvals and EASR registration. A Category 3 PTTW would be required for water takings in excess of 400,000 L/day.

The radius of influence of the dewatering operations will depend on whether an interlocking sheet pile system is used surrounding the foundation / subgrade excavation, and the schedule and duration for the excavation and construction works. If a sheet pile system is used, the radius of influence will be relatively limited and negligible settlement of the existing culvert or embankment is anticipated. If a sheet pile system is not used, some settlement of the soft surficial clayey silt or loose portions of the silt to sand deposit may occur associated with the groundwater drawdown. However, as the surficial clayey silt is thin and discontinuous and the silt to silty sand deposit is also relatively thin, such settlement is estimated to be less than 25 mm and more likely on the order of 15 mm or less at the west end of the existing culvert and the existing highway embankment side slope. Settlement monitoring is not considered necessary for the dewatering operation based on the conditions at this site.

6.6.4 Construction Materials Based on Analytical Testing

The results of analytical testing completed on one sample of the silty sand deposit are summarized in Section 4.3 and presented in Appendix C. The potential for sulphate attack and corrosion are discussed below. However, it is the responsibility of the designer to determine the appropriate construction materials, including the exposure class and ensuring that all aspects of CSA A23.1-24 Section 4.1.1 “*Durability Requirements*” are followed when designing concrete elements.

The potential for sulphate attack on concrete was determined by comparing analytical test results to CSA A23.1-14 Table 3 “*Additional Requirements for Concrete Subjected to Sulphate Attack*”. The water-soluble sulphate concentration measured in silty sand is below 0.1%, which is below the exposure class of S-3 (Moderate) and is considered “Negligible” as per Table 7.2 of the MTO Gravity Pipe Guidelines (2014). Therefore, based on the test results from the sample, the effects of the sulphates may not need to be considered when the designer is selecting the exposure class for the structure. However, consideration should be given to the de-icing salts which may be used surrounding the building when selecting the exposure class.

The silty sand sample measured a pH value of 7.9 and a resistivity value of 13,000 ohm-cm. According to the MTO Gravity Pipe Guidelines, the pH is not considered detrimental to structure durability. The resistivity is greater than 10,000 ohm-cm, which indicates that the soil corrosiveness is less than “Very Low” (10,000 ohm-cm < R < 6,000 ohm-cm), as per Table 3.2 “*Soil Corrosiveness and Resistivity*” of the MTO Gravity Pipe Guidelines (2014).

6.6.5 Piezometer Decommissioning

As noted in Section 6.9.3, it is recommended that the groundwater level at the site be measured closer to the time of construction, in order for the Contractor to assess the dewatering / surface water infiltration flow diversion requirements during construction. The piezometer installed in Borehole RW-1 should be decommissioned during construction and a Non-Standard Special Provision (NSSP) should be added to the Contract Documents; an example NSSP for this purpose is attached in Appendix E.

7.0 CLOSURE

This Foundation Design Report was prepared by Anastasia Poliacik, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., an MTO Foundations Designated Contact and Principal of Golder, conducted an independent technical and quality control review of the report.

Signature Page

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REFERENCES

Canadian Standards Association (CSA). 2019. *Canadian Highway Bridge Design Code (CHBDC, 2019) and Commentary on CAN/CSA-S6-19*. CSA Special Publication.

Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

CSA Group. 2014. A23.1-14/A23.2-14 - Concrete materials and methods of concrete construction / Test methods and standard practices for concrete.

Ministry of Transportation, Ontario. 2014. *Gravity Pipe Design Guidelines*.

Ministry of Transportation, Ontario. 2008. *RSS Wall Design Guidelines*.

ASTM International:

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Ontario Provisional Standard Drawings:

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements
OPSD 3190.100	Walls, Retaining and Abutments, Walls

Ontario Provincial Standard Specifications:

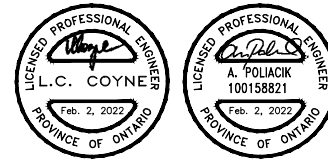
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling – Structures
OPSS.PROV 1010	Construction Specification for Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)

Table 1 - Comparison of Retaining Wall Type and Foundation Options







Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Shallow Foundations – Concrete Cantilever Wall	Feasible and is the preferred foundation option if a retaining wall solution is selected for this site	<ul style="list-style-type: none">• Conventional excavation and construction techniques, particularly as the wall can be constructed prior to widening of the Highway 400 embankment in this area and hence would not require significant excavations or protection systems.• Relatively limited depth of excavation to achieve the required geotechnical resistance.	<ul style="list-style-type: none">• Dewatering of silt to sandy silt deposit required for construction of the footing in “dry” conditions. Sumps/pumps will not be adequate and an eductor system is likely to be required, although there may still be some practical challenges in drawing the water level sufficiently below the subgrade where the underlying till is present at higher elevations; some local sump treatment would be required in such areas.	<ul style="list-style-type: none">• Second lowest cost (higher cost relative to RSS wall, but lower cost compared to deep foundation options)	<ul style="list-style-type: none">• Conventional risks associated with dewatering of silt to silty sand underlain at relatively shallow depth by till deposit, resulting in loose/disturbed subgrade areas; however, this can be mitigated by contractor design of appropriate dewatering system and subexcavation and replacement of unsuitable soils at subgrade level, where applicable
Shallow Foundations - Retained Soil System (RSS) Wall	Feasible, but less advantageous compared to Shallow Foundations - Concrete Cantilever Wall, and would require approval by MTO's RSS Committee	<ul style="list-style-type: none">• Conventional excavation and construction techniques, particularly as the reinforced soil mass can be constructed concurrently with the placement of fill for the westward widening of the Highway 400 embankment.• Shallower excavation as compared with concrete retaining wall option	<ul style="list-style-type: none">• Some risks associated with use of RSS walls adjacent to watercourses particularly where the backfill zone may be submerged via flooding.• Dewatering required for construction of the facing footing and lower portion of the reinforced soil mass in “dry” conditions, although to a lesser extent than that required for deeper excavations for strip footings.	<ul style="list-style-type: none">• Lowest cost	<ul style="list-style-type: none">• Due to adjacent creek/drainage channels, there is potential for flooding which could contribute to loss of granular backfill in the reinforced soil zone. Use of this wall type would require review and approval by MTO's RSS Committee.• Same risk as above related to dewatering to achieve a “dry” subgrade.
Deep Foundations – Driven Piles	Feasible, but not required and less advantageous compared to shallow foundation options	<ul style="list-style-type: none">• Provides higher geotechnical resistances than shallow foundation options, although this is not required for this relatively low-height retaining wall, particularly given that the shallow soils provide adequate bearing resistance for footings.	<ul style="list-style-type: none">• Dewatering required for construction of the pile cap in “dry” conditions, similar to that for a shallow strip footing described above.• Requires more significant working pad for construction compared to shallow foundation options.	<ul style="list-style-type: none">• Third highest cost	<ul style="list-style-type: none">• Risk of piles getting “hung-up” within the very dense till deposit.• Same risk as above related to dewatering to achieve a “dry” subgrade for pile cap
Deep Foundations - Secant pile wall	Feasible, but not required and less advantageous compared to shallow foundation options, particularly given that “top-down” construction is not needed at this site	<ul style="list-style-type: none">• Provides higher geotechnical resistances than shallow foundation options, although this is not required for this wall given that the shallow soils provide adequate bearing resistance for footings.	<ul style="list-style-type: none">• Temporary liners would be required to advance drilled shafts, due to water-bearing non-cohesive soils; appropriate methods would be required to minimize potential for disturbance of soils at base of caisson or soldier pile holes• Requires more significant working pad for construction compared to shallow foundation options.	<ul style="list-style-type: none">• Highest cost	<ul style="list-style-type: none">• Risk of cobble and boulder obstructions within the till deposit.• Risk of disturbance of soils during installation of drilled shafts, requiring temporary liners and tremie concrete techniques
Deep Foundations - Soldier Piles with Reinforced Concrete Facing Panels	Feasible, but not required and less advantageous compared to shallow foundation options, particularly given that “top-down” construction is not needed at this site	<ul style="list-style-type: none">• Provides higher geotechnical resistances than shallow foundation options, although this is not required for this wall given that the shallow soils provide adequate bearing resistance for footings.• Soil anchors could be used to provide additional lateral stiffness to maintain horizontal movement within tolerable limits, if necessary, and these could be maintained within the MTO right-of-way.	<ul style="list-style-type: none">• Likely more time-consuming to install than first three options above due to steps involved (pre-augering for socket holes, placing and concreting the socket of soldier piles, placing backfill in lifts, installing concrete panels and granular drainage layer or pre-fabricated drainage layer).• Groundwater control required for excavations and panel installation.• Requires more significant working pad for construction compared to shallow foundation options.	<ul style="list-style-type: none">• Second highest cost (lower costs compared to secant pile wall, but must also consider groundwater control and drainage costs)	<ul style="list-style-type: none">• Conventional risks associated with dewatering of silt to silty sand underlain at relatively shallow depth by till deposit.
Retained Soil System (RSS) Slope (1.5H:1V)	Feasible, with less excavation/dewatering compared to Shallow Foundations - Concrete Cantilever Wall, but with approval by MTO's RSS Committee required	<ul style="list-style-type: none">• Conventional excavation and construction techniques, particularly as the reinforced soil mass can be constructed concurrently with the placement of fill for the westward widening of the Highway 400 embankment.• Shallower excavation as compared with retaining wall options	<ul style="list-style-type: none">• Some risks associated with use of RSS walls adjacent to watercourses particularly where the backfill zone may be submerged via flooding.• Dewatering required for construction of lower portion of the reinforced soil mass in “dry” conditions, although to a lesser extent than that required for retaining wall options.	<ul style="list-style-type: none">• Similar cost to RSS Wall but less dewatering costs	<ul style="list-style-type: none">• Due to adjacent creek/drainage channels, there is potential for flooding which could contribute to loss of granular backfill in the reinforced soil zone. Use of this slope type would require review and approval by MTO's RSS Committee.• Same risk as above related to dewatering to achieve a “dry” subgrade
Rock Fill Slope (1.5H:1V)	Feasible, with less excavation/dewatering compared to Shallow Foundations - Concrete Cantilever Wall	<ul style="list-style-type: none">• Conventional excavation and construction techniques, particularly as the rock fill placement can be constructed concurrently with the westward widening of the Highway 400 embankment.• Shallower excavation as compared with retaining wall options• Dewatering may not be required.	<ul style="list-style-type: none">• Requires import of rock fill material since rock fill is not readily available in the area.• Requires placement of geofabric below the rock fill to prevent migration of fines into the rockfill.• Placement of rock fill around culvert would impact future excavation works for rehab/extension/replacement, although such work would be well into the future	<ul style="list-style-type: none">• Similar cost to RSS slope (less dewatering but potentially higher costs of importing material).	<ul style="list-style-type: none">• Rock fill slope may provide vegetation and runoff challenges along the slope face.



KEY PLAN
SCALE

1 0 1 2 km

LEGEND

	Borehole – Current Investigation
	Borehole – Previous Investigation (GEOCRETS 30M-13-214)
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
	WL in piezometer, measured on 02/10/2021
	WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
C36-1	310.5	4871168.2	298317.5
RW-1	310.6	4871174.9	298297.8
RW-2	310.3	4871154.9	298295.0

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by MH, drawing file no. X117116615Base (1).dwg, received June 15, 2021.
Topography plan provided in digital format by MH, drawing file no. X117116615Contours.dwg, received June 2, 2021.
GA arrangement provided in digital format by MH, drawing file no. S03.dwg, received January 6, 2022.

[illegible]



Photograph 1: Existing Berm Access Facing Southwest



Photograph 2: Northeast of SWMP Facing South

CLIENT

Morrison Hershfield Limited

CONSULTANT



YYYY-MM-DD January 2022

TAKEN BY

CHECKED BY

PROJECT

**Proposed Retaining Wall, Station 18+800
Highway 400 Widening from North of King Road
To South of Lloydtown-Aurora Road, King City, Ontario
Assignment No. 2017-E-0016-015, G.W.P. 2835-02-00**

TITLE

Site Photographs

PROJECT No.

1786658 (W015)

FIGURE

1

APPENDIX A

Borehole Records

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (<i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (<i>i.e.</i> , some sand)
≤ 10	trace (<i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q_t*), porewater pressure (*u*) and sleeve friction (*f_s*) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, w _L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
Y	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

- Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
- SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
- SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index $= (w_L - w_P)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_P) / I_P$
I_c	consistency index $= (w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{a(e)}$	secondary compression index
C_a	rate of secondary compression
$C_{a(e)}$	modified secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

* Density symbol is ρ . Unit weight symbol is γ .
where $\gamma = \rho \cdot g$ (i.e., mass density multiplied by
acceleration due to gravity)

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (compressive strength)/2

PROJECT		09-1111-0018		RECORD OF BOREHOLE No C36-1		SHEET 1 OF 1		METRIC									
G.W.P.		2835-02-00		LOCATION		N 4871168.2 ; E 298317.5		ORIGINATED BY									
DIST		Central HWY 400		BOREHOLE TYPE		D-50 Track Mount, 108 mm Diameter Solid Stem Augers		COMPILED BY									
DATUM		Geodetic		DATE		November 25, 2010		CHECKED BY									
								SMM									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES											
310.5		GROUND SURFACE															
0.0		TOPSOIL		1A	SS	3											
0.3		Silty sand, trace gravel (FILL) Very loose Brown Moist		2B													
309.4		CLAYEY SILT, trace sand, trace gravel, trace organics Soft to very stiff Brown and black Moist		2A	SS	16											
1.1				2B													
308.3		SILT, trace to some sand, trace clay Compact Brown and grey Moist		3	SS	17											
2.2																	
		SILT and SAND, trace gravel, trace clay (TILL) Compact to very dense Brown to grey below 5.0 m Wet to moist below 3.7 m		4	SS	16											
				5	SS	10											
				6	SS	41											
				7	SS	51											
				8	SS	105											
303.9		END OF BOREHOLE															
6.6		NOTES: 1. Borehole caved at a depth of 3.0 m (Elev. 307.5 m) upon completion of drilling. 2. Water level in open borehole at a depth of 2.1 m (Elev. 308.4 m) upon completion of drilling.															

PROJECT		1786658 (W015)		RECORD OF BOREHOLE No RW-1		SHEET 1 OF 1		METRIC									
G.W.P.		2835-02-00		LOCATION		N 4871174.9; E 298297.8 MTM NAD 83 ZONE 10 (LAT. 43.980538; LONG. -79.581054)		ORIGINATED BY KC									
DIST		Central HWY 400		BOREHOLE TYPE		Power Auger; 203 mm O.D. Hollow Stem Augers		COMPILED BY CC									
DATUM		HT2 0 (Geodetic)		DATE		December 4, 2020		CHECKED BY AMP									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	γ	GR	SA	SI	CL
311.0	GROUND SURFACE																
0.0	SILTY SAND (SM), trace gravel, trace rootlets and organics to 1.4 m depth Loose to compact Brown Moist to wet		1	SS	4		310						OC=1.1%				
	-Wet below 1.5 m depth		2	SS	8												
			3	SS	16		309										
308.8	SILT (ML), trace sand Compact Brown Wet		4	SS	21		308							0	5	95	0
2.2			5	SS	25												
307.3	SILT (ML) and sand, trace clay, trace gravel (TILL) Compact to very dense Brown Moist		6	SS	29		307										
			7	SS	49		306							4	43	46	7
	- Auger grinding between 5.5 m and 6.1 m depth																
	- Slightly plastic between 5.8 m and 7.0 m depth		8	SS	52		305										
	- Auger grinding at 7.0 m depth						304										
			9	SS	78		303										
							302										
			10	SS	91		301							3	36	51	10
300.8	CLAYEY SILT (CL), some sand, trace gravel (TILL) Hard Grey Moist																
10.2			11	SS	90		300										
299.7	END OF BOREHOLE																
11.3	NOTES: 1. Water encountered at a depth of 1.5 m (Elev. 309.5 m) during drilling. 2. Piezometer installed in separate borehole adjacent Borehole RW-1. 3 Water level measured in piezometer as follows: Date Depth (m) Elev. (m) 10-Feb-21 0.6 310.4																

GTA-MTO 001 S:\CLIENTS\MTOWHY_400_KING_TO_LLOYDTOWN02_DATA\GINT\HWY_400_KING_TO_LLOYDTOWN.GPJ GAL-GTA.GDT 5/5/21

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

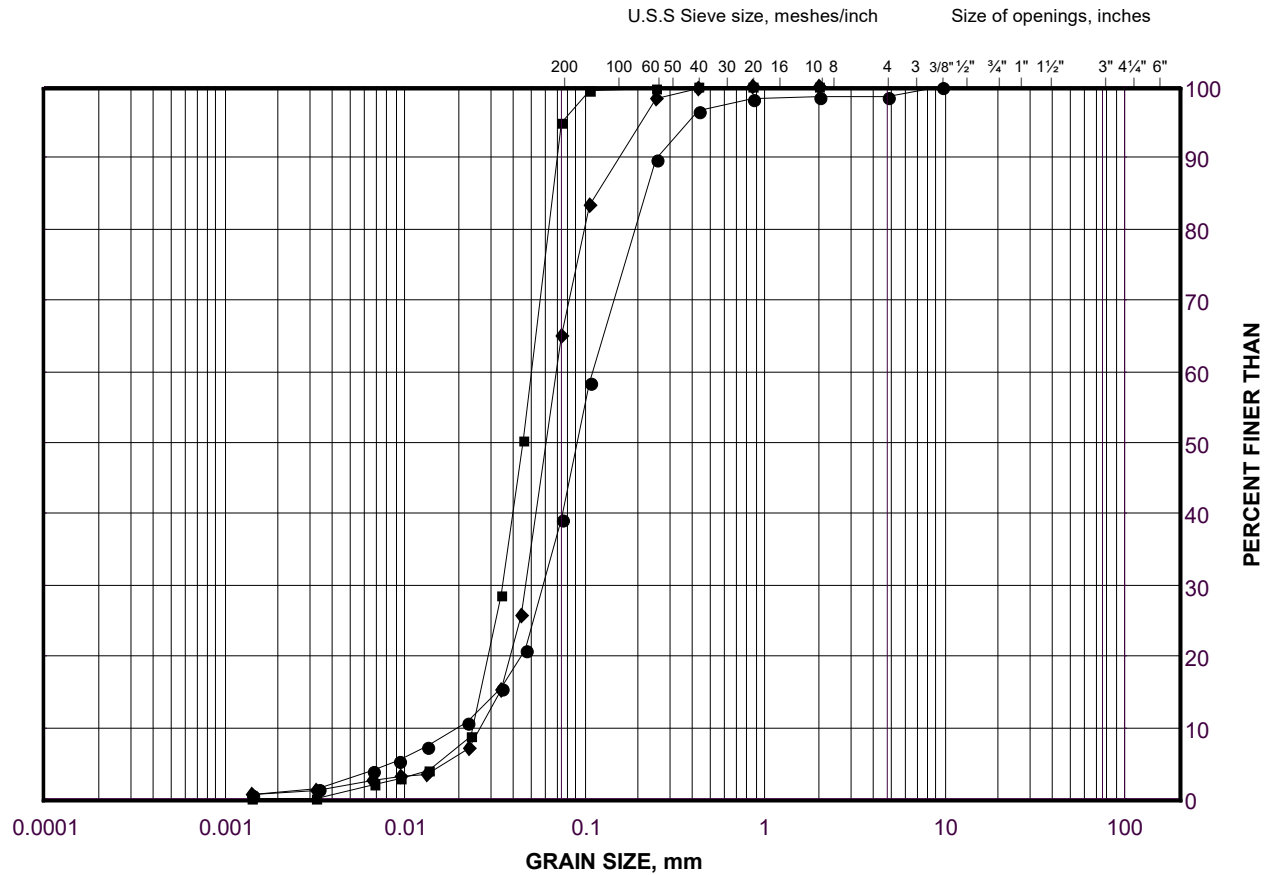
APPENDIX B

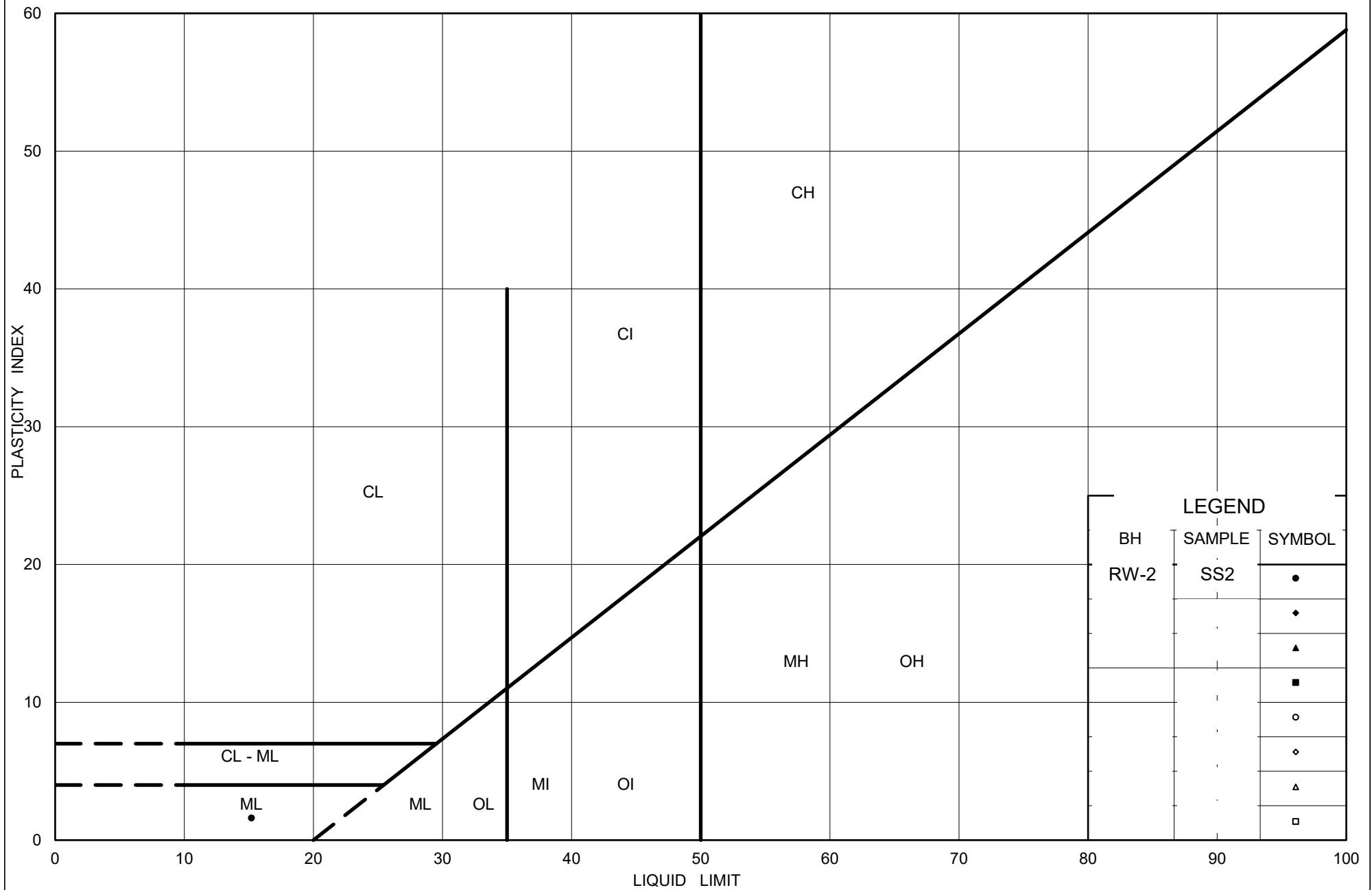
Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

SILT (ML) to SILTY SAND (SM)

FIGURE B-1





Ministry of Transportation

Ontario

PLASTICITY CHART **SILTY SAND (SM) of Slight Plasticity**

Figure No. B-2

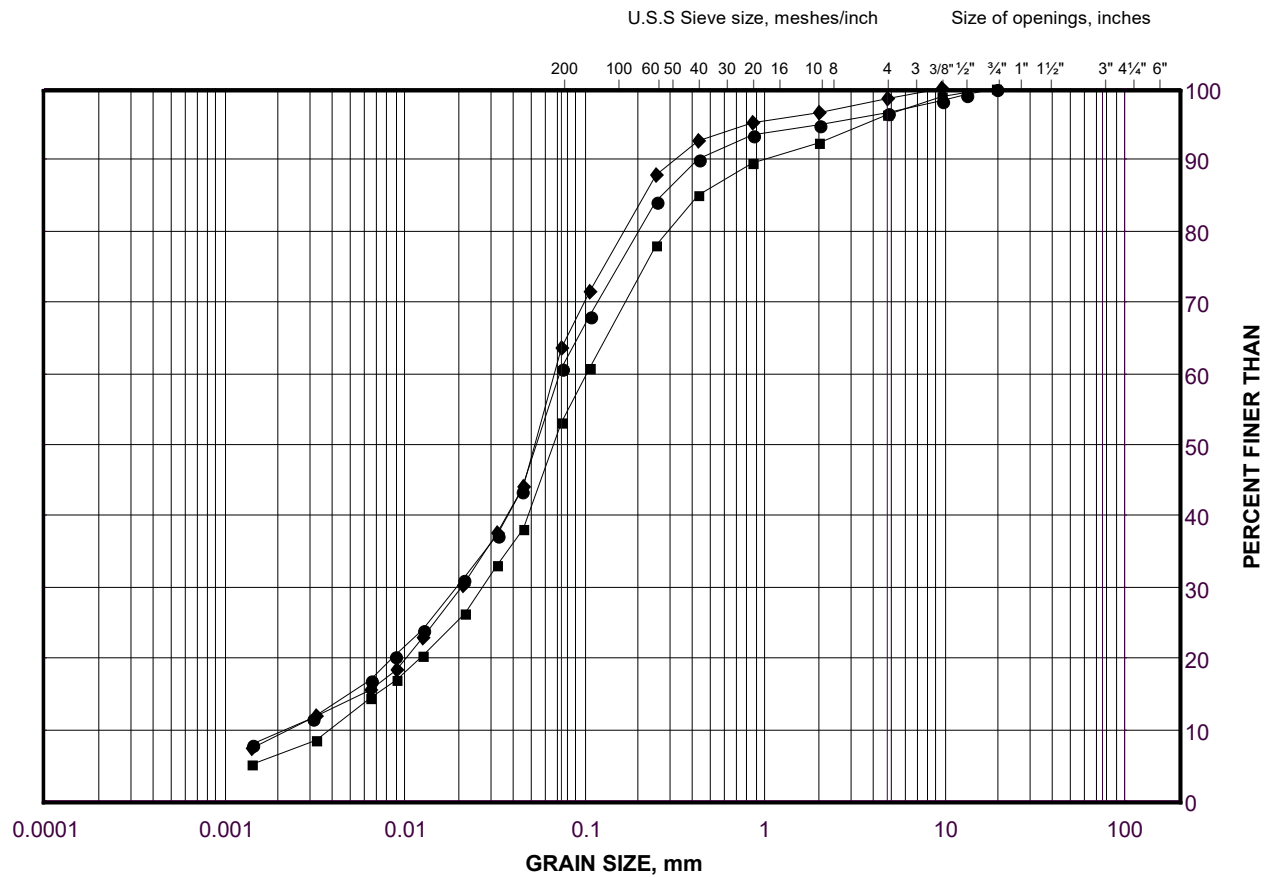
Project No. 1786658-WO15 (RW)

Checked By: AMP

GRAIN SIZE DISTRIBUTION

Sandy SILT (ML) to CLAYEY SILT (CL) (TILL)

FIGURE B-3



LEGEND

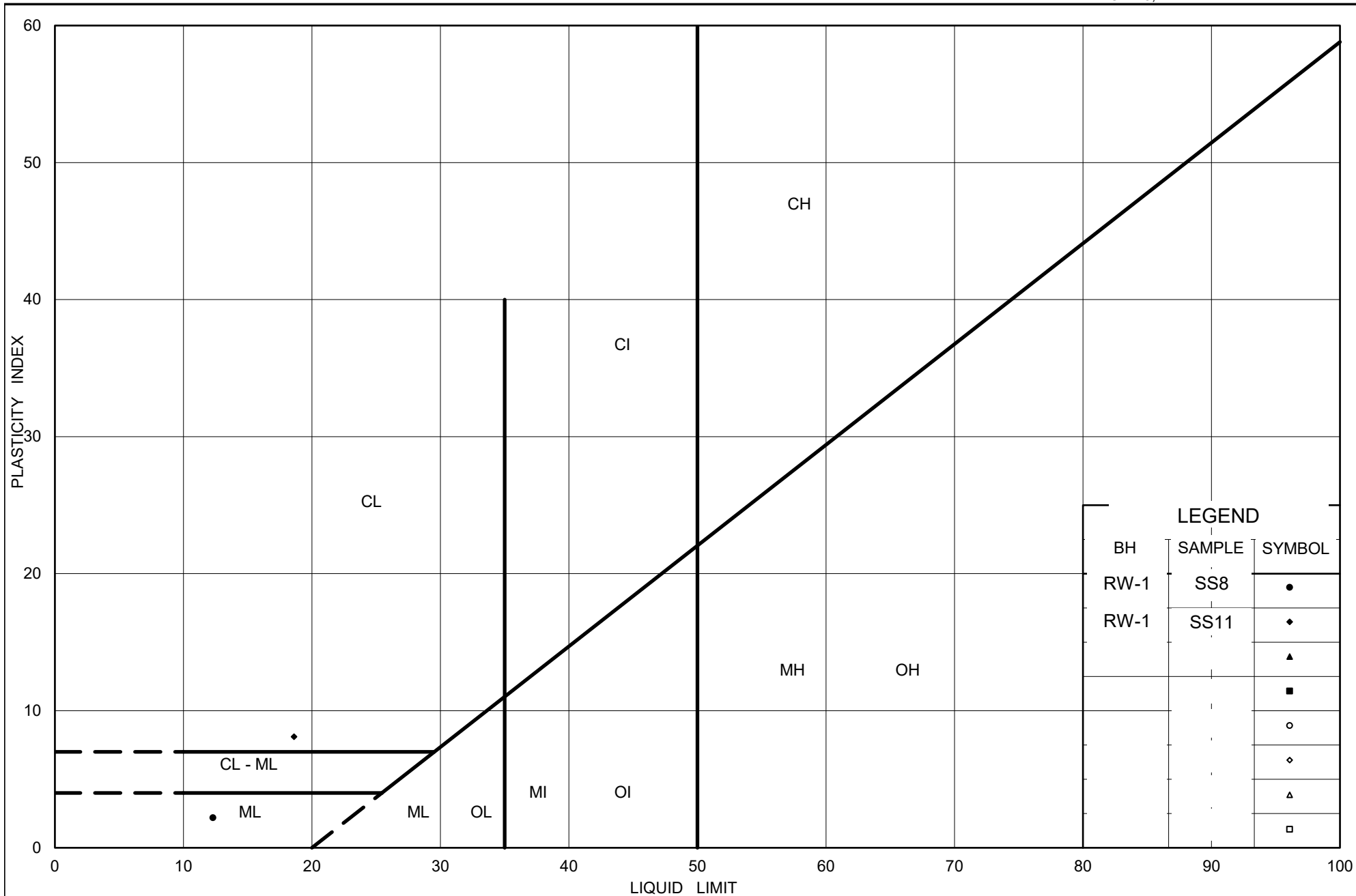
SYMBOL	Borehole	SAMPLE	ELEVATION (m)
●	RW-1	SS10	301.5
■	RW-1	SS7	306.1
◆	RW-2	SS9	302.8

Project Number: 1786658-WO15 (RW)

Checked By:AMP

Golder Associates

Date: 14-Apr-21



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Ontario

PLASTICITY CHART

Sandy SILT (ML) to CLAYEY SILT (CL) (TILL)

Figure No. B-4

Project No. 1786658-WO15 (RW)

Checked By: AMP

APPENDIX C

Analytical Laboratory Tests Results



Your Project #: 1786658 WO 15
Your C.O.C. #: 805726-01-01, 794544-05-01

Attention: Carter Comish

Golder Associates Ltd
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2021/01/28
Report #: R6497762
Version: 4 - Revision

CERTIFICATE OF ANALYSIS – REVISED REPORT

BV LABS JOB #: C0X1766

Received: 2020/12/11, 18:34

Sample Matrix: Soil
Samples Received: 1

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	1	2020/12/17	2020/12/18	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	1	2020/12/17	2020/12/17	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	1	2020/12/16	2020/12/16	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	1	2020/12/14	2020/12/17	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	1	2020/12/17	2020/12/18	CAM SOP-00464	EPA 375.4 m

Remarks:

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Bureau Veritas profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Bureau Veritas in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Bureau Veritas liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Bureau Veritas has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Bureau Veritas, unless otherwise agreed in writing. Bureau Veritas is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.

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Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.



Your Project #: 1786658 WO 15
Your C.O.C. #: 805726-01-01, 794544-05-01

Attention: Carter Comish

Golder Associates Ltd
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2021/01/28
Report #: R6497762
Version: 4 - Revision

CERTIFICATE OF ANALYSIS – REVISED REPORT

BV LABS JOB #: C0X1766

Received: 2020/12/11, 18:34

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Ema Gitej, Senior Project Manager

Email: emese.gitej@bureauveritas.com

Phone# (905)817-5829

=====

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.



BUREAU
VERITAS

BV Labs Job #: COX1766
Report Date: 2021/01/28

Golder Associates Ltd
Client Project #: 1786658 WO 15
Sampler Initials: CC

SOIL CORROSIVITY PACKAGE (SOIL)

BV Labs ID		OKC176			OKC176		
Sampling Date		2020/12/04			2020/12/04		
COC Number		805726-01-01			805726-01-01		
	UNITS	RW-1 SA3	RDL	QC Batch	RW-1 SA3 Lab-Dup	RDL	QC Batch
Calculated Parameters							
Resistivity	ohm-cm	13000		7108299			
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g	<20	20	7114805	<20	20	7114805
Conductivity	umho/cm	78	2	7114634			
Available (CaCl2) pH	pH	7.85		7112629			
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	7114979			
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate							



BUREAU
VERITAS

BV Labs Job #: COX1766
Report Date: 2021/01/28

Golder Associates Ltd
Client Project #: 1786658 WO 15
Sampler Initials: CC

TEST SUMMARY

BV Labs ID: OKC176
Sample ID: RW-1 SA3
Matrix: Soil

Collected: 2020/12/04
Shipped:
Received: 2020/12/11

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7114805	2020/12/17	2020/12/18	Alina Dobreanu
Conductivity	AT	7114634	2020/12/17	2020/12/17	Tarunpreet Kaur
pH CaCl2 EXTRACT	AT	7112629	2020/12/16	2020/12/16	Neil Dassanayake
Resistivity of Soil		7108299	2020/12/17	2020/12/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7114979	2020/12/17	2020/12/18	Alina Dobreanu

BV Labs ID: OKC176 Dup
Sample ID: RW-1 SA3
Matrix: Soil

Collected: 2020/12/04
Shipped:
Received: 2020/12/11

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7114805	2020/12/17	2020/12/18	Alina Dobreanu



BUREAU
VERITAS

BV Labs Job #: COX1766
Report Date: 2021/01/28

Golder Associates Ltd
Client Project #: 1786658 WO 15
Sampler Initials: CC

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	5.7°C
-----------	-------

Revised report (2021/01/28): Split report as per client request.

Results relate only to the items tested.



**BUREAU
VERITAS**

BV Labs Job #: COX1766

Report Date: 2021/01/28

QUALITY ASSURANCE REPORT

Golder Associates Ltd

Client Project #: 1786658 WO 15

Sampler Initials: CC

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
7112629	Available (CaCl ₂) pH	2020/12/16			100	97 - 103			0.080	N/A
7114634	Conductivity	2020/12/17			102	90 - 110	<2	umho/cm	1.6	10
7114805	Soluble (20:1) Chloride (Cl ⁻)	2020/12/18	118	70 - 130	103	70 - 130	<20	ug/g	NC	35
7114979	Soluble (20:1) Sulphate (SO ₄)	2020/12/18	115	70 - 130	106	70 - 130	<20	ug/g	NC	35

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).



BUREAU
VERITAS

BV Labs Job #: COX1766
Report Date: 2021/01/28

Golder Associates Ltd
Client Project #: 1786658 WO 15
Sampler Initials: CC

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Anastassia Hamanov, Scientific Specialist

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.



CHAIN OF CUSTODY RECORD

Page of

INVOICE TO: Company Name: #1326 Golder Associates Ltd Attention: Accounts Payable Address: 6925 Century Ave Suite 100 Mississauga ON L5N 7K2 Tel: (905) 567-4444 Fax: (905) 567-6561 Email: CanadaAccountsPayableInvoices@golder.com			REPORT TO: Company Name: Golder Associates Attention: Darcy Hansen Address: ccornish@golder.com Tel: (905) 567-4444 Ext-2064 Fax: _____ Email: Darcy.Hansen@golder.com			PROJECT INFORMATION: Quotation #: B80683 P.O. #: _____ Project: 1786658 WO 15 Project Name: _____ Site #: _____ Sampled By: _____			Laboratory Use Only: BV Labs Job #: _____ Bottle Order #: _____ COC #: _____ Project Manager: _____ C#805726-01-01 Ema Gitej												
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BV LABS DRINKING WATER CHAIN OF CUSTODY						ANALYSIS REQUESTED (PLEASE BE SPECIFIC)						Turnaround Time (TAT) Required: Please provide advance notice for rush projects									
Regulation 153 (2011) <input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input checked="" type="checkbox"/> Medium/Fine <input type="checkbox"/> Table 2 <input checked="" type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input checked="" type="checkbox"/> Table 3 <input type="checkbox"/> Agri/Other <input type="checkbox"/> For RSC <input type="checkbox"/> Table _____			Other Regulations <input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> Reg 558 <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> MISA Municipality _____ <input type="checkbox"/> PWQO <input type="checkbox"/> Reg 406 Table _____ <input type="checkbox"/> Other _____			Special Instructions O. Reg 347 Schedule 4			Field Filtered (please circle): Metals / Hg / Cr / V Corrosivity eq (Cl, SO4, pH, EC/Relativity)						Regular (Standard) TAT: (will be applied if Rush TAT is not specified): Standard TAT = 5-7 Working days for most tests. Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details. Job Specific Rush TAT (if applies to entire submission) Date Required: _____ Time Required: _____ Rush Confirmation Number: _____ (call lab for #)						
Include Criteria on Certificate of Analysis (Y/N)? <input checked="" type="checkbox"/>																					
Sample Barcode Label		Sample (Location) Identification		Date Sampled	Time Sampled	Matrix															
1 RW-1 SA3				Dec 4 2020	AM	Soil															
2 TISB-1 SA3				Nov 26 2020																	
3 CCTV-2 SA3B				Nov 27 2020																	
4 CCTV-4 SA4				Nov 13 2020																	
5 CCTV-5 SA3				Nov 10 2020																	
6 Z6-6 SA4B				Nov 9 2020																	
7 Z7-5 SA2				Nov 9 2020																	
8 Z8-6 SA#6				Nov 17 2020																	
9 Z9-5 SA12				Nov 5 2020																	
10 Z3-5 SA8				Nov 12 2020	V	V															
* RELINQUISHED BY: (Signature/Print)				Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)				Date: (YY/MM/DD)	Time	# jars used and not submitted		Laboratory Use Only							
Carter Cornish				20/12/11	6pm	Pn/ALEXANDRA FORBES				20/12/11	18:29			Time Sensitive		Temperature (°C) on Receipt		Custody Seal		Yes	No
														47.6 °C		Present					
																Intact					

11-Dec-20 18:34
 Ema Gitej

COX1766
 AF2 ENV-736

* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BV LABS' STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVLABS.COM/TERMS-AND-CONDITIONS.

* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVLABS.COM/RESOURCES/CHAIN-OF-CUSTODY-FORMS.



Bureau Veritas Laboratories
6740 Campbell Road, Mississauga, Ontario Canada L5N 2L8 Tel: (905) 817-5700 Toll-free 800-563-6266 Fax: (905) 817-5777 www.bvlabs.com

CHAIN OF CUSTODY RECORD

Page of

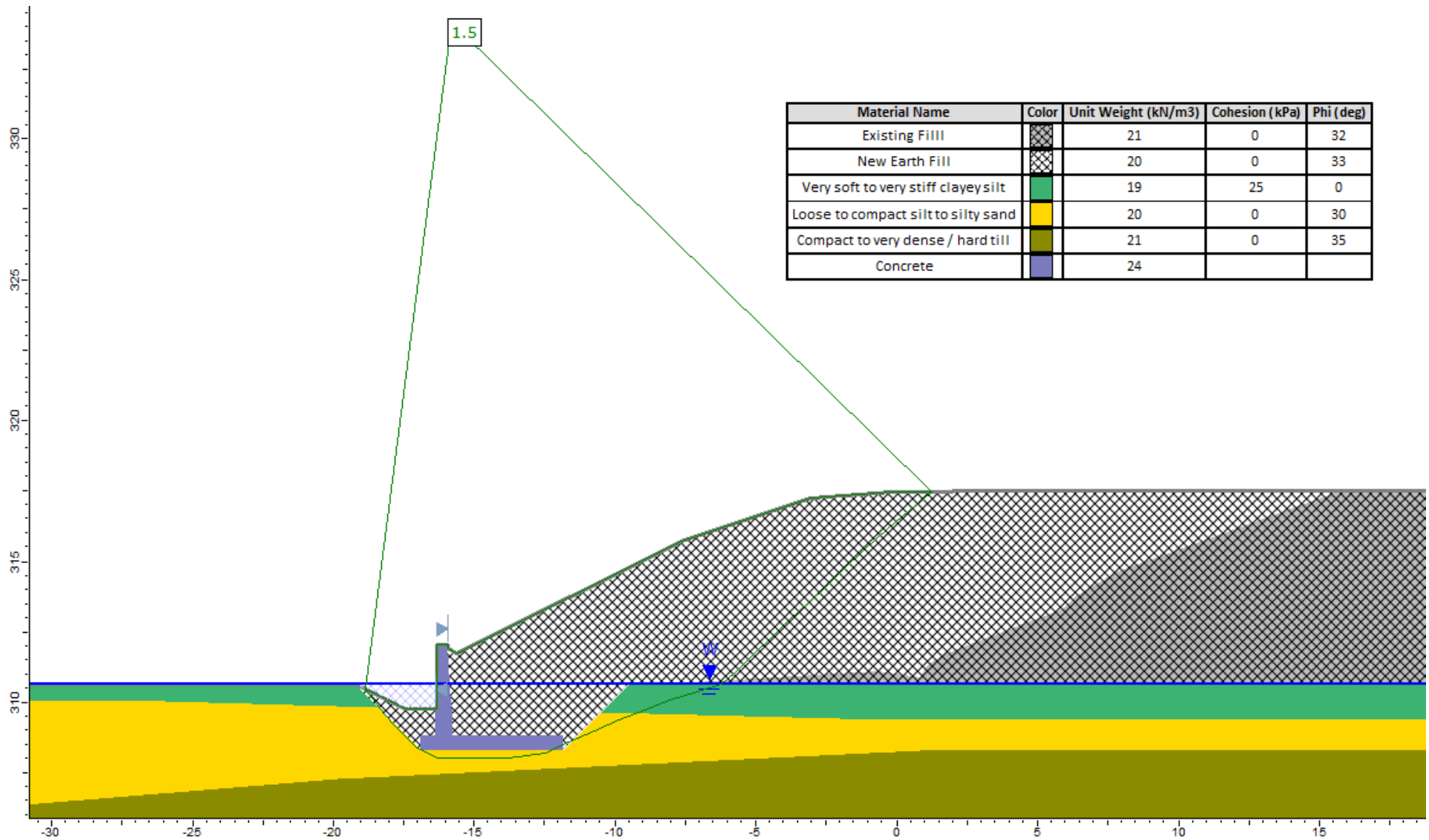
INVOICE TO:		REPORT TO:		PROJECT INFORMATION:		Laboratory Use Only:	
Company Name: <u>1326</u> <u>#2292</u> Golder Associates Ltd		Company Name: <u>Kimberley Rose</u> <u>aka Carter Comish</u>		Quotation #: <u>B80683</u>		BV Labs Job #: <u>1895923.4000</u> <u>178658 WWS</u>	
Attention: <u>Accounts Payable</u>		Attention: <u>Kimberley Rose</u>		P.O. #:		Bottle Order #: <u>794544</u>	
Address: <u>100 Scotia Ct</u> <u>6725 Century Ave #100</u>		Address: <u>ccomish@golder.com</u>		Project: <u>1895923.4000</u> <u>178658 WWS</u>		COC #:	
Tel: <u>(905) 723-2727</u>		Tel: <u>(905) 723-5491 Ext 6644</u>		Project Name:		Project Manager: <u>Erna Gitej</u>	
Fax: <u>(905) 723-2182</u>		Fax: <u>(905) 723-2182</u>		Site #:		Turnaround Time (TAT) Required:	
Email: <u>CanadaAccountsPayableInvoices@golder.com</u>		Email: <u>Kimberley_Rose@golder.com</u>		Sampled By:		Please provide advance notice for rush projects	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BV LABS DRINKING WATER CHAIN OF CUSTODY				ANALYSIS REQUESTED (PLEASE BE SPECIFIC)			
Regulation 153 (2011)		Other Regulations		Special Instructions		Regular (Standard) TAT:	
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input checked="" type="checkbox"/> Medium/Fine		<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw		<u>O. Reg 347</u>		(will be applied if Rush TAT is not specified)	
<input type="checkbox"/> Table 2 <input checked="" type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse		<input type="checkbox"/> Reg 558 <input type="checkbox"/> Storm Sewer Bylaw		<u>aka</u>		Standard TAT = 5-7 Working days for most tests.	
<input checked="" type="checkbox"/> Table 3 <input type="checkbox"/> Agri/Other <input type="checkbox"/> For RSC		<input type="checkbox"/> MISA <input type="checkbox"/> Municipality		<u>Schedule 4</u>		Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.	
<input type="checkbox"/> Table		<input type="checkbox"/> PWQO <input type="checkbox"/> Reg 406 Table				Job Specific Rush TAT (if applies to entire submission)	
		<input checked="" type="checkbox"/> Other				Date Required: _____ Time Required: _____	
Include Criteria on Certificate of Analysis (Y/N)? <u>Y</u>						Rush Confirmation Number: _____ (call lab for #)	
Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle):	# of Bottles	
1 <u>34-5 SA 6</u>		<u>Nov 12 2020</u>			Metals / Hg / Cr VI	Comments	
2 <u>35-4 SA 6</u>		<u>Nov 12 2020</u>			<u>Geog 153 VOCS by HS & F & T (6ml)</u>		
3 <u>36-5 SA 9</u>		<u>Nov 2 2020</u>			<u>Reg 153 PPHs</u>		
4					<u>Reg 153 Metals & Inorganics - PPH</u>		
5					<u>corrosivity pH</u>		
6					<u>short term</u>		
7							
8							
9							
10							
* RELINQUISHED BY: (Signature/Print)		Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
<u>Kimberley Rose</u>		<u>20/11/12</u>	<u>6p</u>	<u>Kimberley Rose</u>	<u>20/12/11</u>	<u>18:34</u>	
Laboratory Use Only		Time Sensitive		Temperature (°C) on Receipt		Custody Seal	
				<u>4/7/16</u>		Present	
						Intact	
						Yes	
						No	
* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BV LABS' STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVLABS.COM/TERMS-AND-CONDITIONS.				SAMPLER MUST BE KEPT COOL (< 10°C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BV LABS			
* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.				White: BV Labs Yellow: Client			
** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVLABS.COM/RESOURCES/CHAIN-OF-CUSTODY-FORMS.							

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APPENDIX D

Global Stability Figures

Global Stability Analysis Concrete Cantilever Wall Temporary (Undrained) Analysis

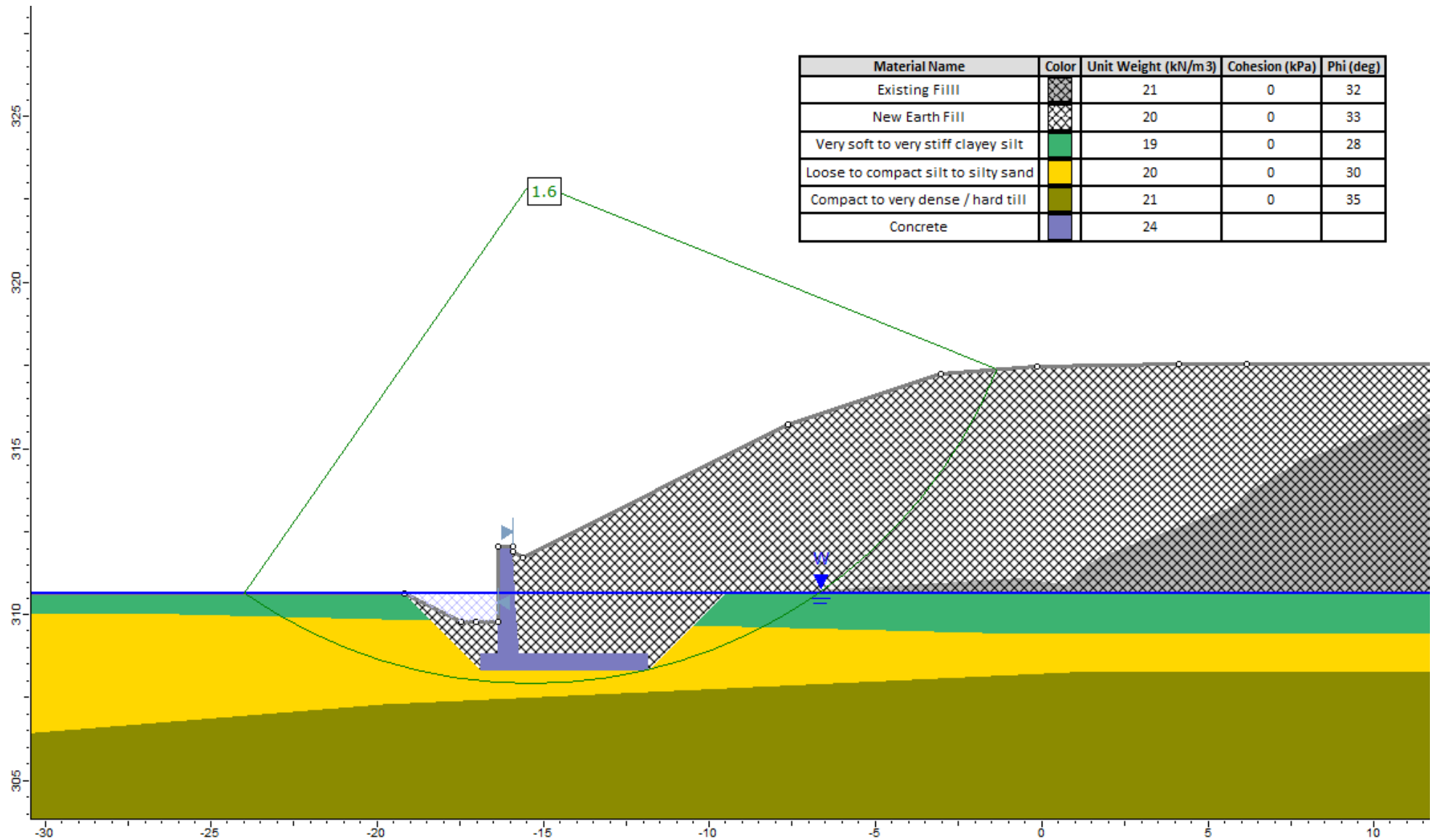


Global Stability Analysis

Concrete Cantilever Wall

Long Term (Drained) Analysis

Figure D-2

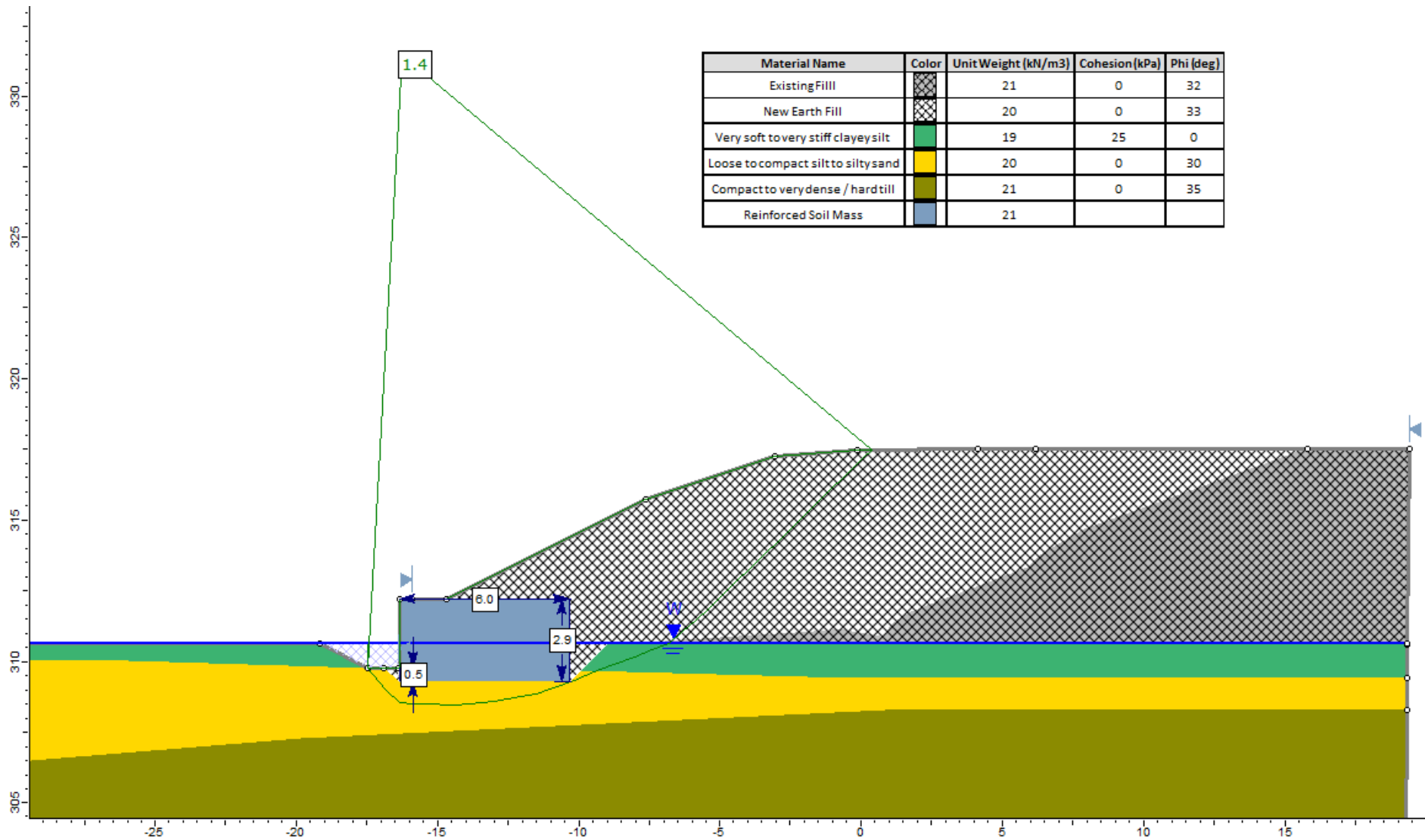


Global Stability Analysis

RSS Wall

Temporary (Undrained) Analysis

Figure D-3

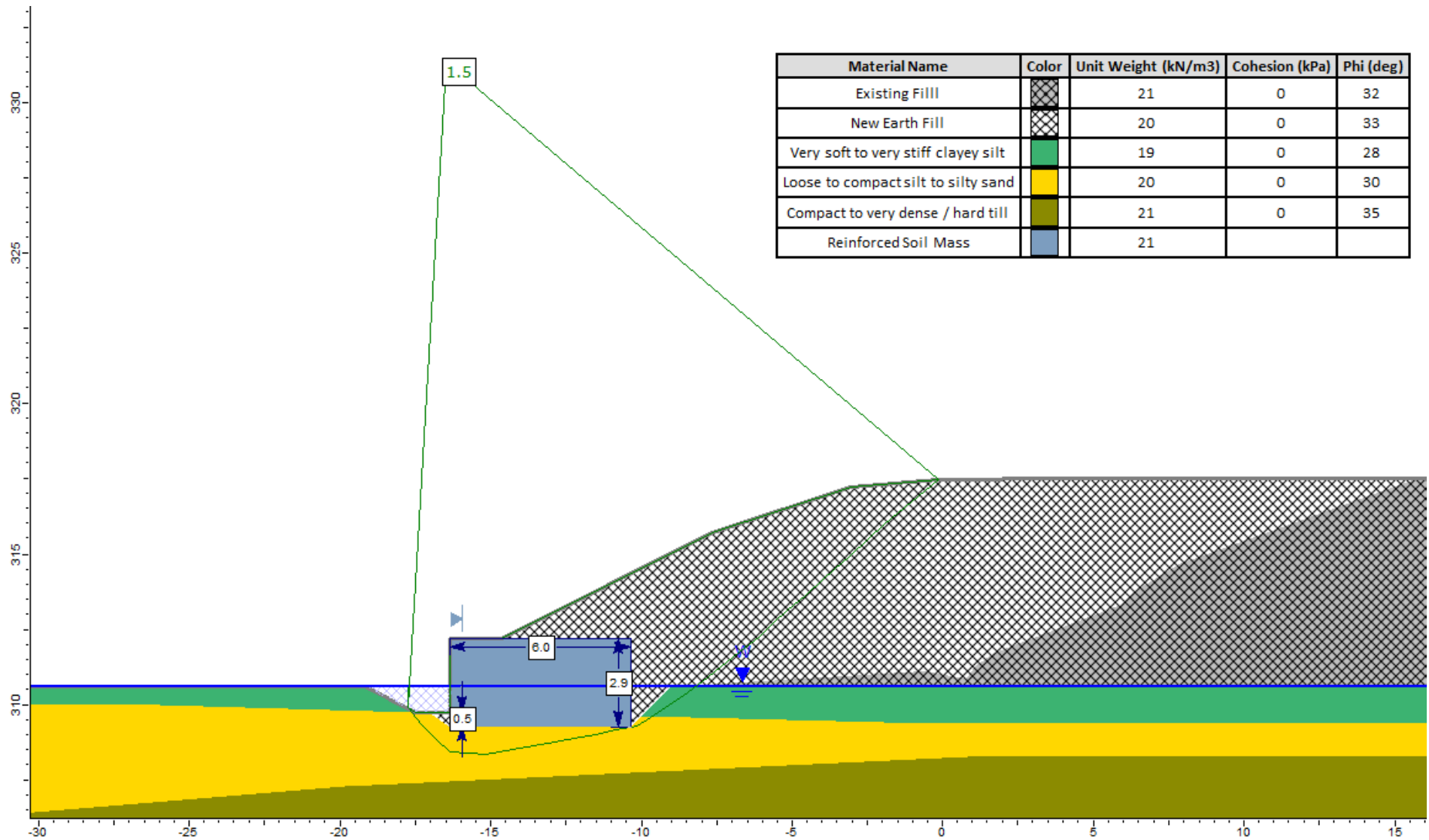


Global Stability Analysis

RSS Wall

Long Term (Drained) Analysis

Figure D-4

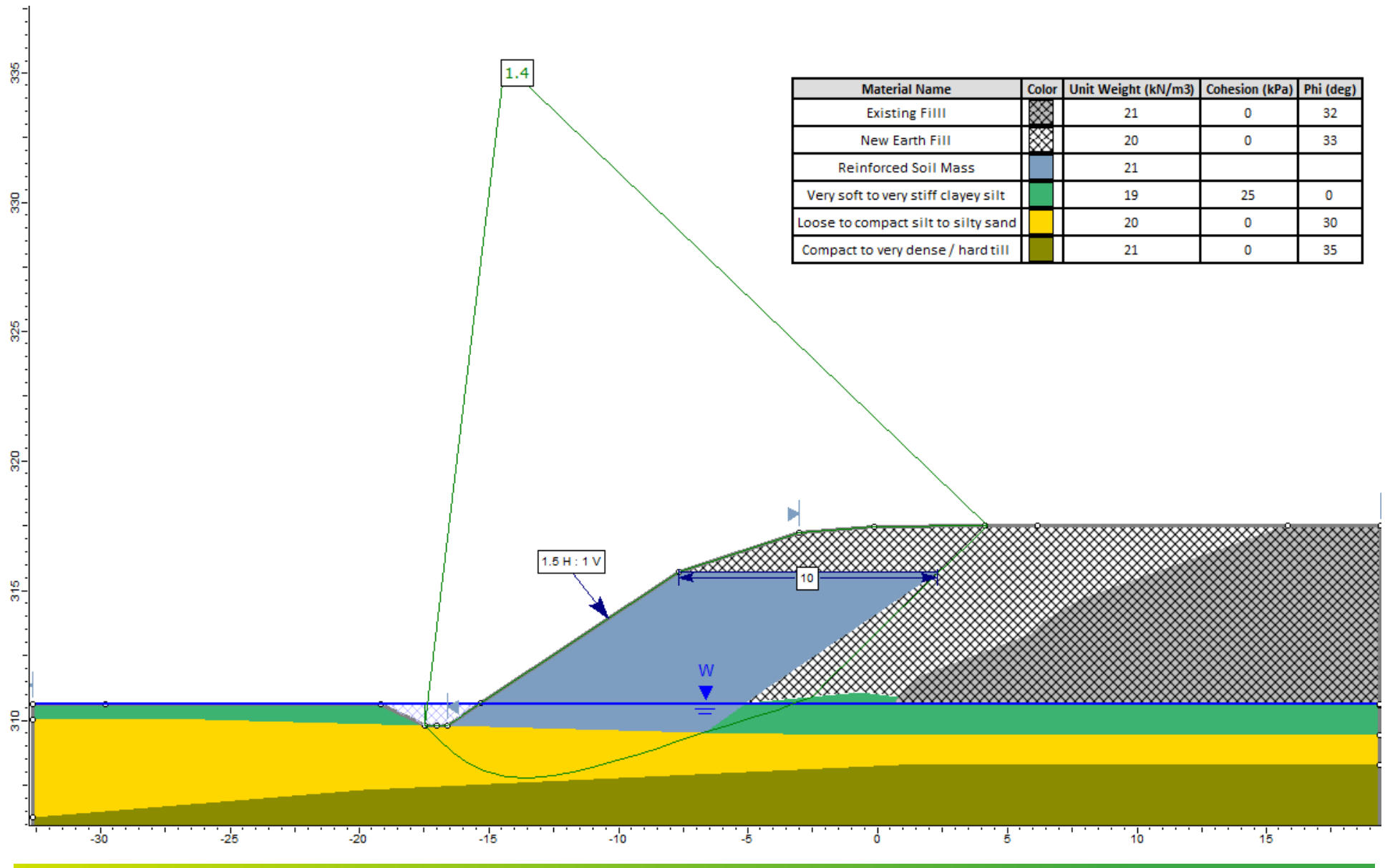


Global Stability Analysis

RSS Slope

Temporary (Undrained) Analysis

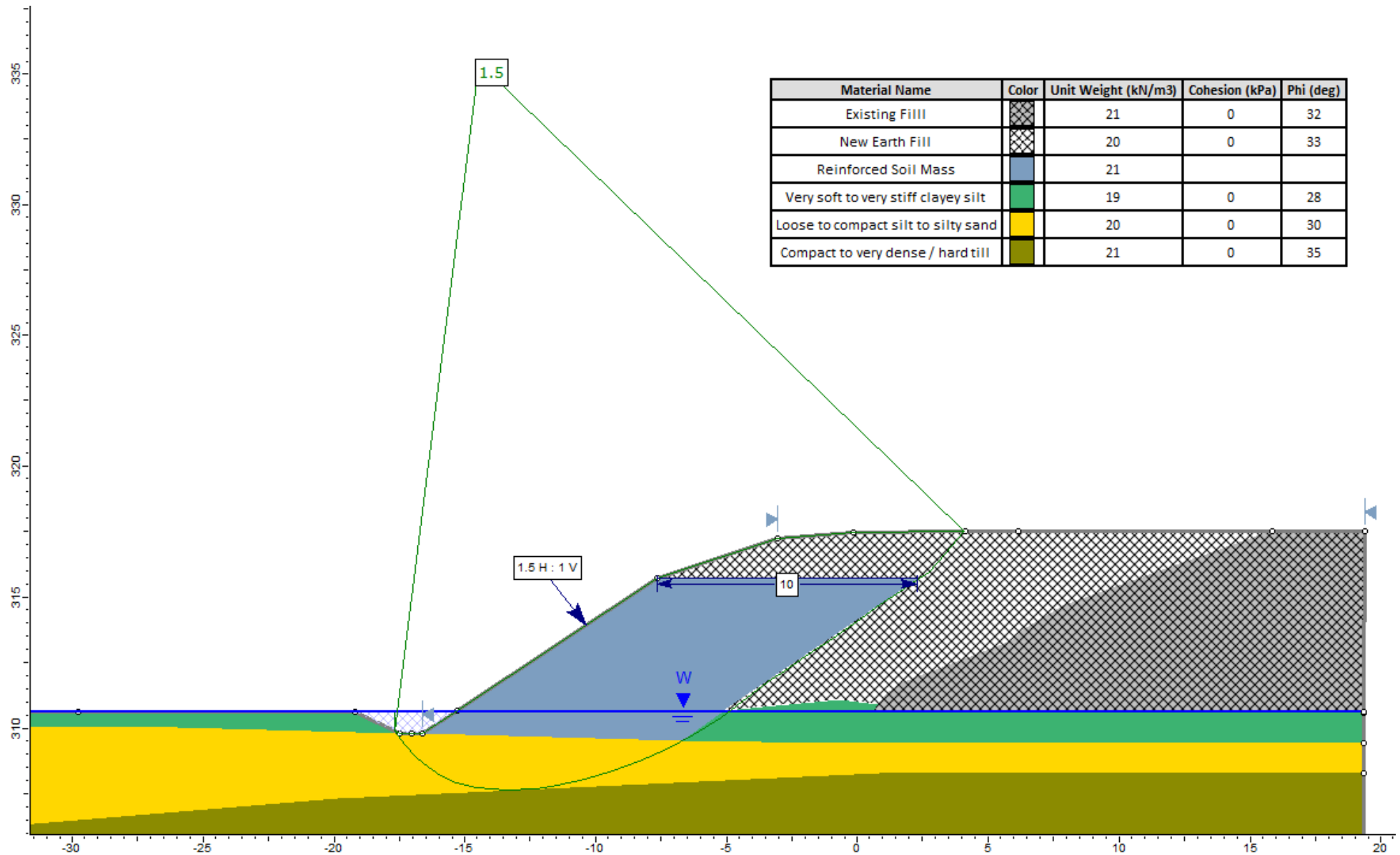
Figure D-5



Global Stability Analysis

RSS Slope

Long Term (Drained) Analysis

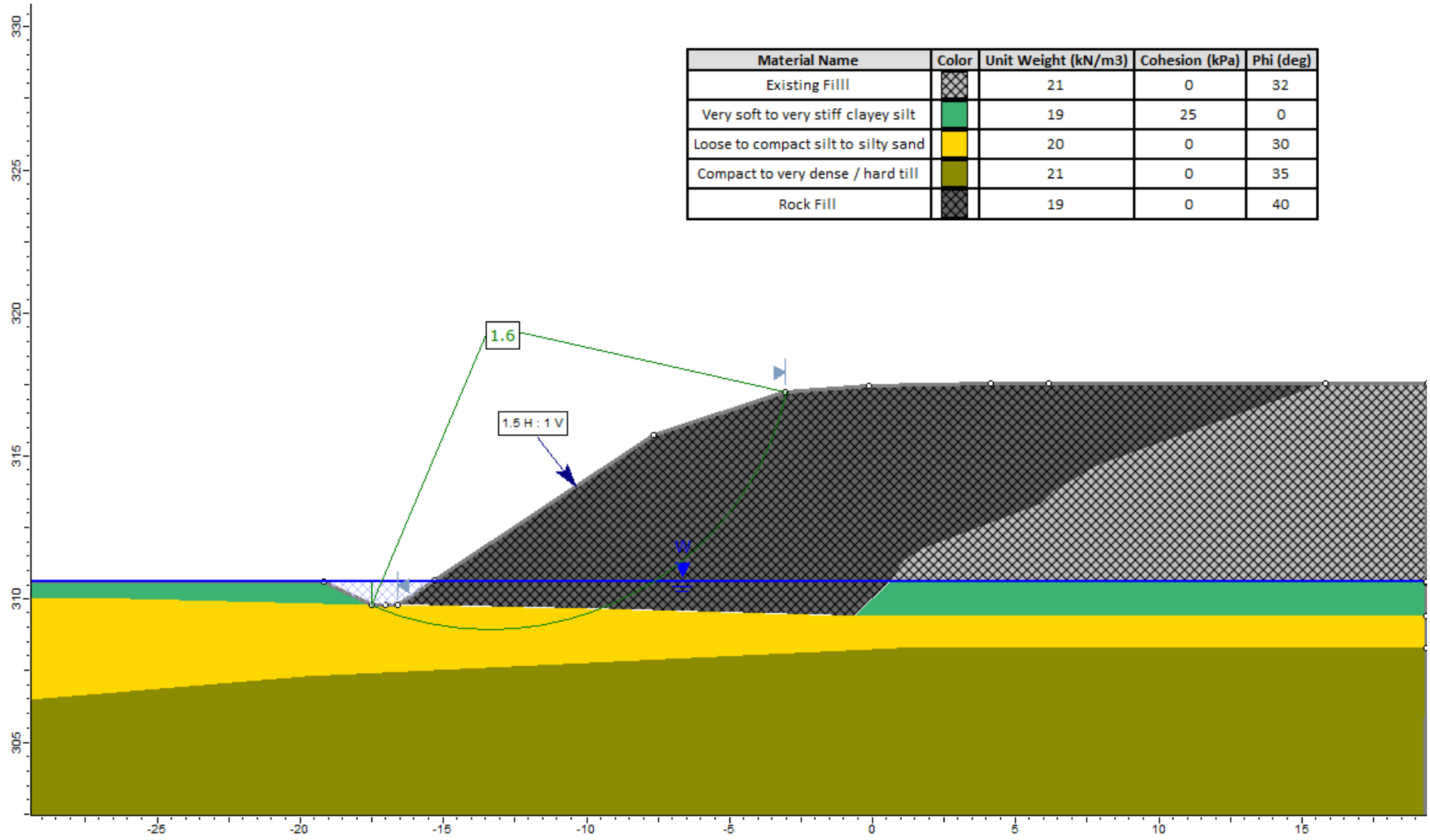


Global Stability Analysis

Rock Fill Slope

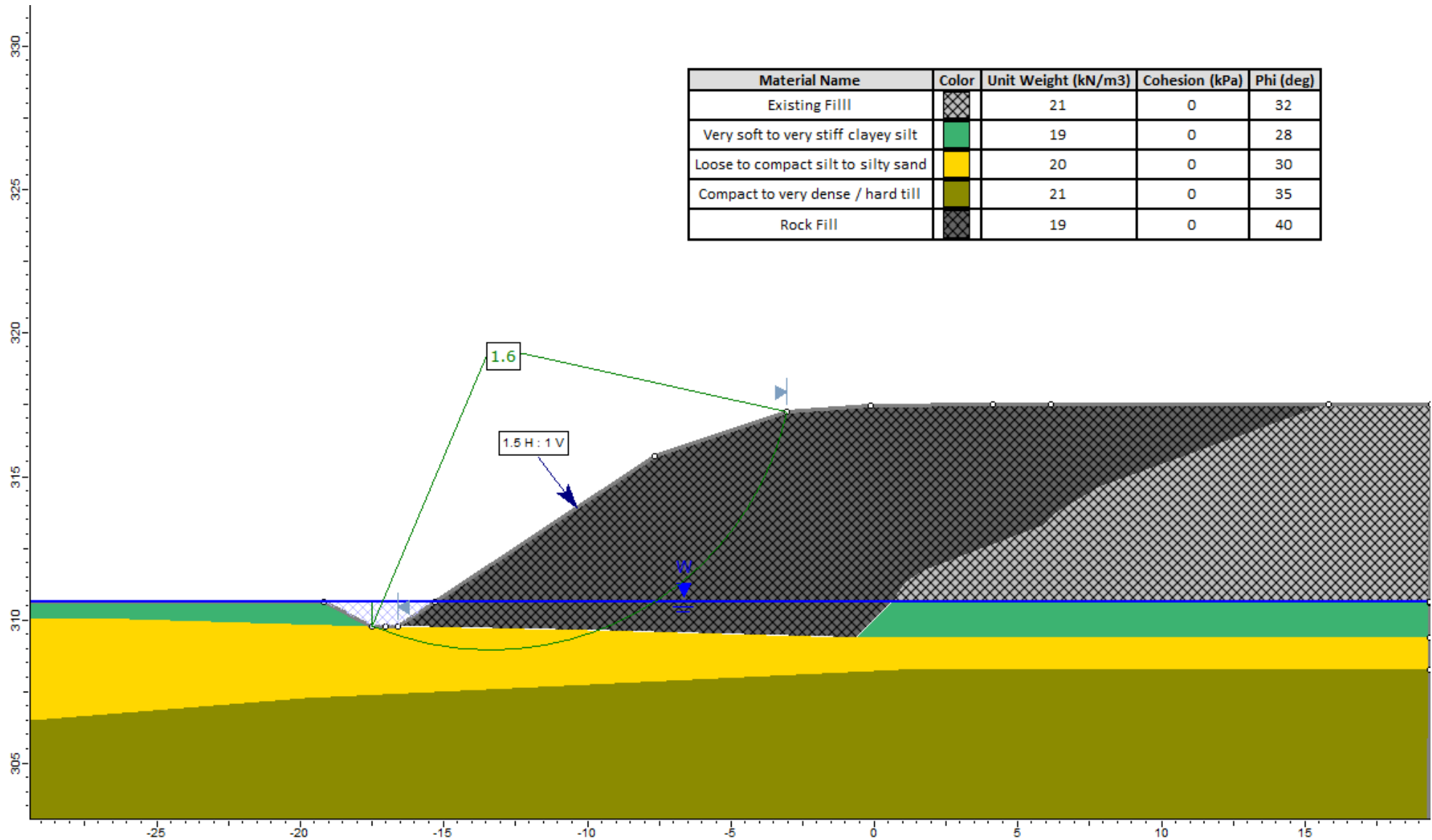
Temporary (Undrained) Analysis

Figure D-7



Global Stability Analysis Rock Fill Slope Long Term (Drained) Analysis

Figure D-8



APPENDIX E

Special Provision

WORKING SLAB – Item No.

Special Provision

1.0 SCOPE

This special provision covers the requirements for the supply and placement of a concrete working slab under structure foundations and for trenchless installation shaft construction.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications, or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling - Structure

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – Not Used

5.0 MATERIALS

Concrete for working slabs shall have a minimum 28-day strength of 20 MPa.

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavations for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Within four hours following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade, as specified in the Contract Documents.

7.03 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

Payment at the Contract price for this tender item shall be full compensation for all labour, Equipment and Materials to do the work.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN0003

March 8, 2018

Amendment to OPSS 902, November 2010

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517 Dewatering
OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 2-year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of any adjacent properties, buildings, underground structures, water wells, utilities, and structures within a distance of 50 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

DECOMMISSIONING OF PIEZOMETERS – Item No.

Special Provision

1.0 SCOPE

This special provision covers the requirements for the decommissioning of the piezometers located within the vicinity of the proposed retaining wall.

A Standpipe piezometer was installed in Borehole RW-1. The piezometer has been left in place to allow for monitoring of groundwater levels up to the time of construction. The piezometer location (relative to MTM NAD 83 Zone 10 and in latitude and longitude), piezometer diameter, borehole diameter, and piezometer depth are summarized below.

Standpipe Piezometer Identification	Approximate Location		PVC Pipe and Screen diameter / Borehole diameter	Depth (Below Ground Surface) to Tip of Screen
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
RW-1	4,871,174.9 (43.980538)	298,297.8 (-79.581054)	50 mm / 203 mm	4.5 m

2.0 REFERENCES – Not Used

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – Not Used

5.0 MATERIALS – Not Used

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

As part of the construction activities, the Contractor shall properly decommission the standpipe piezometers prior to the start of the construction works. The abandonment / decommissioning method for standpipe piezometers shall satisfy at least the minimum requirements of Ontario Regulation 903 Wells, as amended under the Ontario Water Resources Act.

In addition, the Contractor shall provide a written record of the decommissioning procedure to the Contract Administrator. The record shall include plugging material used, depth of plugging material and limit of the PVC standpipe/screen removal.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

Payment at the Contract price for this tender item shall be full compensation for all labour, Equipment and Materials to do the work.



golder.com